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INVESTIGATION OF CONTAINMENT AREA DESIGN TO MAXIMIZE HYDRAULIC --ETC(U)

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DREDGED MATERIAL RESEARCH PROGRAM

TECHNICAL REPORT D-78-12

INVESTIGATION OF CONTAINMENT AREA DESIGN TO MAXIMIZE HYDRAULIC EFFICIENCY

by

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May 1978

Final Report

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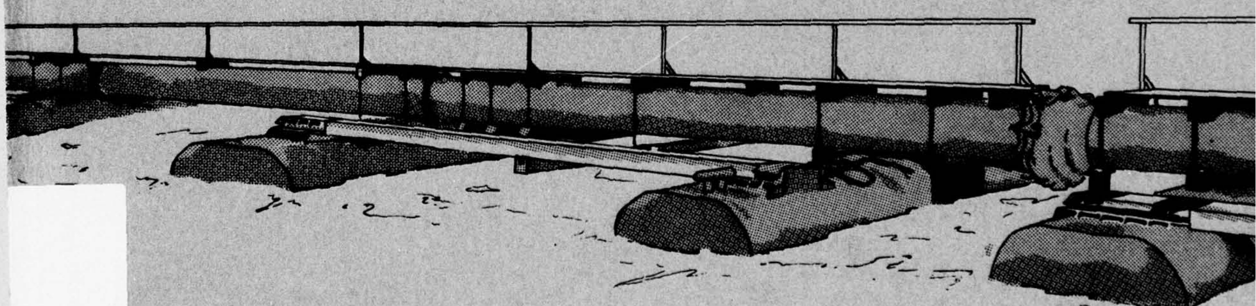

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SUBJECT: Transmittal of Technical Report D-78-12

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1. The report transmitted herewith represents the results of one of the research efforts (work units) initiated as part of Task 2C (Containment Area Operations) of the Corps of Engineers' Dredged Material Research Program (DMRP). Task 2C was included as part of the Disposal Operations Project, which among other considerations included research into the various ways of improving the efficiency and acceptability of facilities for confining dredged material on land.

2. Confining dredged material on land is a disposal alternative to which practically no specific design or construction improvement investigations (much less applied research) had been addressed prior to the DMRP. Being a form of waste-product disposal, dredged material placement on land has seldom been evaluated on other than purely economic grounds with emphasis nearly always on lowest possible cost. There has been a dramatic increase within the last few years in the amount of land disposal necessitated by confining dredged material classified as polluted. Attention necessarily is directed more and more to the environmental consequences of this disposal alternative and methods for minimizing adverse environmental impacts.

3. Several DMRP work units were conducted to investigate and improve facility design and construction and to investigate concepts for increasing facility capacities for both economic and environmental purposes. The study reported herein, conducted by Bryan J. Gallagher and Company, was conducted to investigate methodologies for improving the hydraulic efficiencies of dredged material containment areas and to develop general guidelines for the proper design and operation of containment areas in their inlet and outlet arrangements. The study consisted of:

a. A review of published literature and technical reports.

b. Site visits and field tests at 10 active disposal areas to obtain recent operational data.

c. Development of a mathematical model and computer programs to predict flow patterns and retention times for different containment area configurations.

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d. Formulation of a general methodology for the design of efficient containment areas.

4. During the site visits and field tests, information on operational data, cost factors, effluent standards, existing guidelines, problem areas, and present and future needs concerning disposal area requirements was compiled through interviews with District personnel and on-site inspections, tests, and sampling. The range of settling effectiveness at all sites ranged from 88 to 99 percent solids removal. The settling performance data correlated reasonably well with the factors controlling retention time and demonstrated the importance of ponding water over dredged material in a containment basin. Dye-dispersion tests showed that short-circuiting and wind effects reduce the hydraulic efficiencies in open-water basins to 50 percent or less of hydraulic efficiency under typical conditions. Predicted retention-time distributions for these basins based on the hydraulic model were less than ideal plug-flow retention times but were considerably higher than actual measured times. This was attributed to strong wind effects. Wind-induced circulation was also determined on a theoretical basis to be a dominant factor.

5. It was concluded that the addition of spur dikes to increase the effective length-to-width ratio prevents short-circuiting between inlet and outlet, retards wind-induced circulation, and is the most economical method of maximizing hydraulic efficiency. Other recommendations include the specification of minimum ponding depths based on selected withdrawal principles and the design of long rectangular weirs to prevent flow concentrations and resuspension problems.

6. This study is but one of several studies addressing the problems of increasing the efficiency of containment areas. Guidelines presented herein should be considered interim with the final guidelines to be contained in a synthesis report on sizing and operating containment areas that is being developed from the results of this and the other related studies.

John L. Cannon

JOHN L. CANNON
Colonel, Corps of Engineers
Commander and Director

Unclassified

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REPORT DOCUMENTATION PAGE

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This study was conducted to investigate methodologies for improving the hydraulic efficiencies of dredged material containment areas and to develop general guidelines for the proper design and operation of containment areas and their inlet and outlet arrangements. The study consisted of (a) a review of published literature and technical reports, (b) site visits and field tests at ten active disposal areas to obtain recent operational data, (c) development of a mathematical model and computer programs to predict flow → over

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20. ABSTRACT (Continued).

patterns and retention times of different containment area configurations, and (d) formulation of a general methodology for the design of efficient containment areas.

The sites visited were confined disposal areas with ongoing operations in the Baltimore, Charleston, Galveston, Mobile, Norfolk, Philadelphia, Portland, Savannah, Seattle, and Vicksburg Corps of Engineers Districts. Information on operational data, cost factors, effluent standards, existing guidelines, problem areas, and present and future needs concerning disposal requirements was compiled through interviews with District personnel and on-site inspections and sampling.

The range of settling effectiveness at all sites extended from 88 to 99 percent removal of solids. The settling performance data correlated reasonably well with the factors controlling retention time and demonstrated the importance of ponding water over dredged material in a containment basin. Dye-dispersion tests conducted at Yazoo City, Mississippi, showed that short-circuiting and wind effects reduce hydraulic efficiencies in open basins to 50 percent or less under typical conditions. Predicted retention-time distributions for these basins based on the hydraulic model were less than ideal plug flow retention times but considerably higher than actual measured times, and this was attributed to strong wind effects. Wind-induced circulation was also determined to be a dominant factor on a theoretical basis.

It was concluded that the addition of spur dikes to increase the effective length-to-width ratio, prevent short-circuiting between inlet and outlet, and retard wind-induced circulation was the most economical method of maximizing hydraulic efficiency, particularly for large, square-shaped areas. Other recommendations include the specification of minimum ponding depths based on selective withdrawal principles and the design of long, rectangular weirs to prevent flow concentration and resuspension problems.

SUMMARY

Environmental considerations have greatly increased the requirement for confined disposal of dredged material in containment areas which also must act as effective sedimentation basins. The lack of proper guidelines for the uniform design of containment areas has resulted in problems being encountered by Corps of Engineers Districts in meeting standards for acceptable quality of disposal area effluents. In some cases, disposal sites are ineffective due to inadequate use of available area for settling or a reduction in retention time caused by short-circuiting problems. These sites are considered hydraulically inefficient and either show a reduced performance or have been greatly oversized to satisfy their requirements. The work reported herein concentrated on methodologies to maximize hydraulic efficiencies of dredged material containment areas and resulted in a set of recommendations for the planning, design, and operation of containment areas to improve their overall performance.

This study included an extensive review of recent dredging and disposal operations to obtain a practical base for the development of improved design recommendations. It was learned that although problems do exist, disposal operations have improved and, in many areas, good solids removal is being achieved, sometimes through extraordinary efforts, and further improvement will be difficult. Many areas are now exceeding 90 percent removal effectiveness but even this may not be sufficient for the future. Occasionally, the performance of some areas has deteriorated to 75 to 85 percent removal under worst-case conditions. When poor performance does occur, it is usually caused by either the inherent limitations of the settling process when effective areas are insufficient (due to poor planning or the lack of suitable areas) or by improper management of sites resulting in very inefficient operations.

The two most important requirements for obtaining reliable settling performance are (a) adequate effective area and retention time to remove solids and (b) proper ponding depths and weir arrangements to prevent the

discharge of removed solids through resuspension problems. Effective areas can be maximized through the addition of spur dikes which will increase the effective length-to-width ratios, decrease short-circuiting, and retard wind-induced circulation problems. Ponding depths should be as high as possible to provide sufficient retention times required. Wind appears to be a dominant factor in causing higher flow velocities that decrease hydraulic efficiency, and larger ponding depths will also help in preventing this problem. Long, rectangular, sharp-crested weirs with low heads are also required to prevent flow concentration and resuspension problems from increased velocities. The principles of selective withdrawal can be applied to obtain weir designs and ponding depths which will not disturb the stability of stratified flow over outfalls, resulting in maximum retention of solids. The selective withdrawal concept is being further investigated and a refined design procedure will be forthcoming in a separate report on weirs, with detailed guidelines for maintaining effluent quality.

Practical recommendations are given to ensure that adequate space and retention time are obtained when planning containment facilities through a simple, systematic procedure combined with methods to maximize the effective area of all available space. The careful use of these recommendations will maximize the hydraulic efficiencies of confined disposal areas with minimum land and cost requirements. Furthermore, proper management of these areas as specified will provide the highest practical levels of effluent quality achievable from gravity sedimentation processes, which will aid in meeting increasingly stringent standards now being imposed. In some cases, the design recommendations will be difficult to implement but are necessary if improved performance is required. The results of this study show that large, well-designed containment areas will be needed in the future if gravity sedimentation processes alone are relied on to meet tight water quality standards. Economic considerations indicate that large square-shaped areas with internal spur dikes are the most economical for maximizing hydraulic efficiency.

PREFACE

The work described in this report was performed under Contract DACW39-76-C-0124, titled "Investigation of Containment Area Design to Maximize Hydraulic Efficiency," dated 29 June 1976, between the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, and Brian J. Gallagher and Company, Inc. The research was sponsored by the Office, Chief of Engineers (DAEN-CWO-M) under the civil works research program, "Dredged Material Research Program (DMRP)." The work was part of the DMRP Disposal Operations Project.

The research was conducted during the period June 1976 to June 1977 under the direction of Brian J. Gallagher, Principal Investigator. Major contributions to the program were made by Drs. G. M. Karadi, R. J. Krizek, R. Y. Lai, and D. K. Atmatzidis. The weir design and selective withdrawal sections were prepared by Dr. Karadi, and Dr. Lai was responsible for the hydraulic model development and applications. Dr. Atmatzidis served as co-editor with Dr. Krizek, who provided overall guidance and assistance. All four investigators contributed invaluably to the development of the final recommendations for improving hydraulic efficiency.

The program was coordinated by Mr. Newton C. Baker and Ms. Marian E. Poindexter, Research Civil Engineers, Environmental Laboratory (EL), WES, who provided much appreciated assistance and direction. Grateful acknowledgement is made to Mr. R. L. Montgomery, Chief, Design and Concept Development Branch (EL), who directly contributed important technical information and assistance. The author also wishes to acknowledge the many staff and engineering personnel of the Districts visited during the study, who provided the practical basis for this work. Mr. C. C. Calhoun, Jr., DOP Manager, together with Dr. John Harrison, Chief, and Dr. R. T. Saucier, Special Assistant for Dredged Material Research, EL, administered the contract. The Directors of WES during the performance of this investigation were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
microns	0.000001	meters
inches	2.54	centimeters
feet	0.3048	meters
miles (U. S. statute)	1.609344	kilometers
square feet	0.09290304	square meters
acres	4046.856	square meters
cubic feet	0.02831685	cubic meters
cubic yards	0.7645549	cubic meters
acre-feet	1233.482	cubic meters
gallons (U. S. liquid)	3.785412	cubic decimeters
feet per second	0.3048	meters per second
miles (U. S. statute) per hour	1.609344	kilometers per hour
cubic feet per second	0.02831685	cubic meters per second
degrees (angular)	0.01745329	radians

INVESTIGATION OF CONTAINMENT AREA DESIGN TO MAXIMIZE HYDRAULIC EFFICIENCY

PART I: INTRODUCTION

1. Environmental considerations have greatly increased the requirement for confining dredged material on land. Consequently, thousands of acres of additional land are required annually throughout the United States for dredged material disposal operations. In order to minimize this need for additional land disposal areas, it is essential that containment structures be properly designed and managed to obtain maximum hydraulic efficiency with minimum area and cost requirements. Furthermore, maximum hydraulic efficiency and retention of dredged sediments will coincide with improved quality of effluents being returned to the natural waterways, with resultant environmental and economic benefits. This is particularly important since new effluent quality standards are being imposed on supernatants released from disposal areas and are likely to become more stringent in the future.

2. The U. S. Army Corps of Engineers has been developing improved methods for dredged material disposal and containment through the Dredged Material Research Program. Several studies have been initiated on basic sedimentation and consolidation theories and on the proper sizing and design of containment structures with respect to bulk storage requirements. However, the design of containment areas based on storage requirements alone may cause inefficient hydraulic performance and result in poor quality of disposal area effluents. Many other factors such as shape of containment area, internal configurations, and arrangement of inlet and outlet devices also play a critical role with respect to hydraulic efficiency. Due to these considerations, this investigation was conducted as part of a combined effort to improve the overall design and operational efficiency of containment structures with respect to removal and retention of solid materials from dredged sediment disposal waters.

Objective

3. The objective of this investigation was to develop general guidelines for use by Corps of Engineers Districts in the planning, design, and management of dredged material containment areas to obtain maximum hydraulic efficiency and removal effectiveness at minimum costs. This report provides practical guidelines and recommendations to Corps of Engineers Districts for both the optimum hydraulic operation of existing disposal areas and improved designs for new areas when required.

Scope and Approach

4. This investigation concentrated on the hydraulic characteristics of existing and new disposal areas and their appurtenances which affect the removal and retention of suspended solids. The investigation was based on an extensive review of existing published literature and Corps of Engineers reports dealing with relevant subject matter. Also, personal observations of present disposal practices and interviews with key personnel were conducted at the following Corps of Engineers Districts: Baltimore, Charleston, Galveston, Mobile, Norfolk, Philadelphia, Portland, Savannah, Seattle, and Vicksburg. Finally, studies were conducted and supported by field tests at actual disposal sites which resulted in the development of computer models useful for synthesizing flow patterns in disposal areas and estimating overall hydraulic efficiencies for optimization purposes. All of this information and data were then integrated to produce the recommendations contained herein.

5. The two major requirements for achieving removal effectiveness in disposal operations are providing proper retention times to allow sufficient settling of solids and then preventing the settled solids from being resuspended and returned to the effluent stream. Maximum hydraulic efficiency is obtained when retention times are maximized with respect to theoretically ideal values. The primary factors affect-

ing retention times are containment area size and shape, total volume, flow patterns and velocities, and inlet and outlet arrangements--all with respect to the particular type, nature, and quantities of sediments being dredged. Therefore the following sections of this report discuss each of the above considerations in addition to a basic discussion of standard practices employed by various Corps of Engineers Districts. Special attention is given to weir design and operation and to the development of a computer model for the prediction of flow patterns and optimum configurations of containment areas. Economic considerations are discussed and final conclusions and recommendations are presented in summary form.

6. It must be emphasized that this study does not deal with the structural integrity of containment structures and devices. Appropriate guidelines should always be used for designing any dike, berm, or device within or through a soil structure such as a discharge pipe. In some areas, recommendations for improving hydraulic efficiency (such as increasing ponding depths) may conflict with structural integrity and safety guidelines. In those cases, the latter guidelines must always be adhered to first.

Structure of Report

7. Part II of the report briefly reviews the background material for the study and Part III summarizes the information and data collected during field visits. Part IV is based on a theoretical analysis of basin hydraulics and presents the results of a computer model which predicts flow patterns for different basin configurations. Part V discusses the release of supernatants and provides general guidelines for weir design and operation. Part VI of the report discusses overall economic considerations of basin design. All parts of the report contain separate conclusion sections while Part VII summarizes the overall conclusions and recommendations for the entire study. Appendices A, B, C, and D provide greater detail and analytical material for parts III, IV, V, and VI, respectively, and Appendix E is a list of symbols and abbreviations used in this report.

PART II: BACKGROUND

8. Dredged material containment facilities are man-made structures which receive dredged bottom sediments in slurry form, retain the suspended solids, and return the clarified waters to the main water body. Thus, the two basic functions of a containment area are (a) to effectively contain all dredged bottom sediments and (b) to release waters of acceptable quality. Furthermore, most containment areas act as ultimate disposal sites for the dredged material, although rehandling and/or beneficial use of the retained solids is occasionally practiced and may become important in the future.

9. With the exception of a small number of areas around the country, the history of confined disposal operations is relatively brief. In the past, the design, construction, and operation of containment areas was oriented primarily toward satisfying maximum storage requirements. In recent years, economic and environmental considerations have emphasized the need for the development of uniform guidelines for dredging and confined disposal operations to achieve consistently superior levels of disposal area performance with respect to suspended solids removal and discharge of environmentally acceptable waters. The practice and problems in the confinement of dredged material have been recently documented (Murphy and Zeigler, 1974). In the following paragraphs the parameters affecting the performance of disposal areas will be discussed on the basis of information extracted from available literature, site visits, and field sampling; and background material will be presented to provide the basis for the study presented in subsequent parts of this report.

Parameters Affecting Disposal Area Performance

10. The major components of a dredged material containment area are shown schematically in Figure 1. A tract of land is surrounded by dikes with a height H_d to form a confining area with size A_d, and the

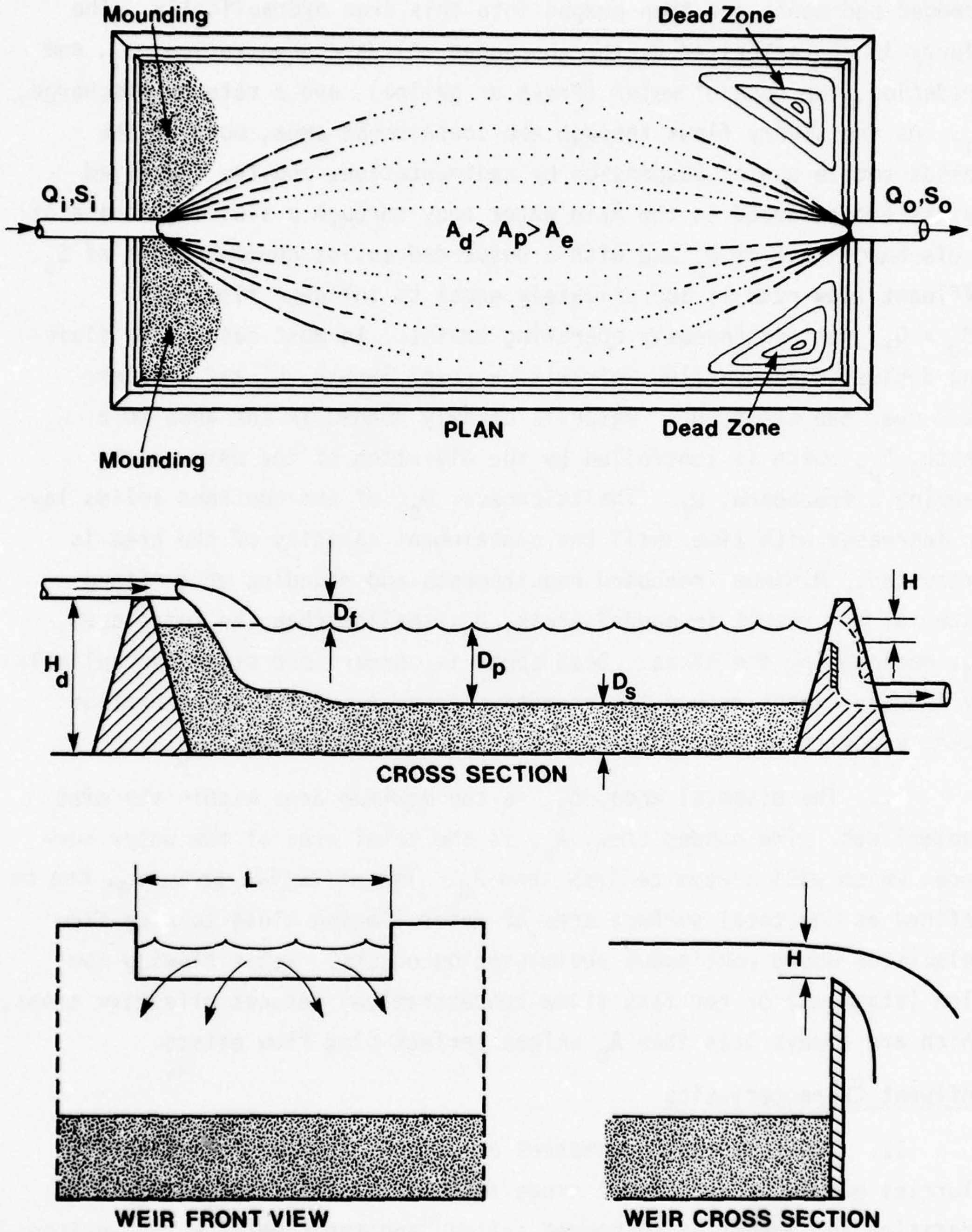


Figure 1. Schematic diagram of dredged material containment area

dredged sediments are then pumped into this area hydraulically. The slurry is characterized by the suspended solids concentration, S_i, and gradation, the type of water (fresh or saline), and a rate of discharge, Q_i. As the slurry flows through the containment area, most of the solids settle out of suspension by sedimentation, and the clarified waters are returned to the main water body through a sluicing device at a discharge rate of Q_o and with a suspended solids concentration of S_o. Effluent flow rate is approximately equal to influent flow rate ($Q_o \simeq Q_i$) for continuously operating basins. In most cases, the sluicing device is an overflow weir with a crest length, L, and a water head over the crest, H. Water is usually ponded in the area to a depth, D_p, which is controlled by the elevation of the weir crest, leaving a freeboard, D_f. The thickness, D_s, of the confined solids layer increases with time until the containment capacity of the area is exhausted. Minimum freeboard requirements and mounding of confined material may result in ponded areas, A_p, smaller than the total area, A_d, enclosed by the dikes. Dead spots in corners and other hydraulically inactive zones reduce the effective area, A_e, where sedimentation takes place to values considerably below the ponded area, A_p.

11. The disposal area, A_d, is the maximum area within the dike centerlines. The ponded area, A_p, is the total area of the water surface, which will always be less than A_d. The effective area, A_e, can be defined as the total surface area of water flowing close to plug flow velocities where continuous sedimentation occurs. Water flowing too slow (stagnant) or too fast (flow concentration) reduces effective areas, which are always less than A_p unless perfect plug flow exists.

Influent Characteristics

12. The important parameters associated with dredged material slurries pumped into disposal areas are the rate of discharge, the gradation and amount of suspended solids, and the type of water environment. Dredging takes place in fresh, brackish, and saline waters, and the density of the water environment affects the natural state of flocculation of the dredged sediments.

13. Discharge Rate. The rate at which dredged material slurries are discharged in disposal areas depends on the type and size of the dredge and the length and diameter of the pipeline used. For continuous pumping, the flow rate, Q_i, in cubic feet per second is given by

$$Q_i = \frac{\pi}{576} (d_p)^2 V_p \qquad (1)$$

where d_p is the pipeline diameter in inches and V_p is the slurry discharge velocity in ft/sec. The relationships between pipeline diameter, discharge velocity, and flow rate are illustrated in Figure 2.

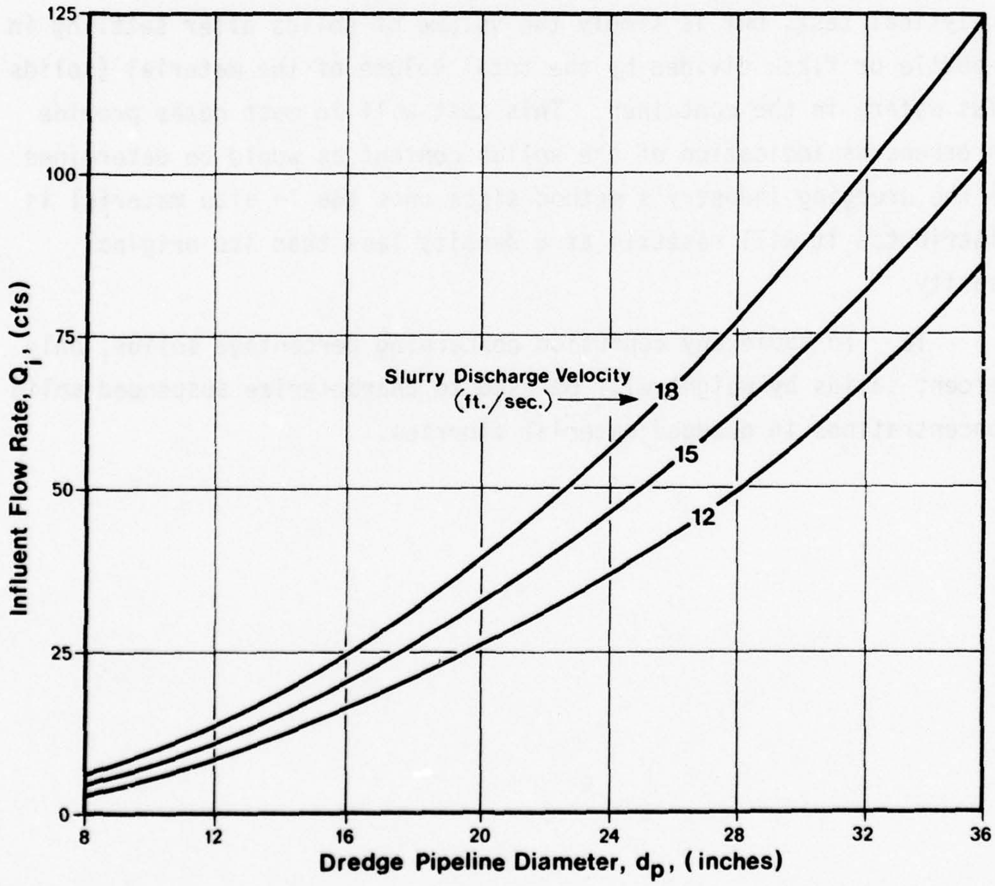


Figure 2. Flow rate as a function of pipe size and flow velocity

14. Suspended Solids Concentration of Influent. The solids content of dredged material slurries may be expressed as the percentage of solids by volume, by apparent volume, or by weight. It is customary in the dredging industry to use percentage of solids by volume since the dredging operation is a volumetric one. The percent solids is based on the in situ volume of the sediment dredged (solids plus water) and not the volume of the solids alone. Therefore, the in situ density of the material must be known in order to determine the true volume of solids.

15. Percent solids is in some instances based on apparent volume. The reported volume of solids is not determined by a standard analytical test, but is simply the volume of solids after settling in a bottle or flask divided by the total volume of the material (solids plus water) in the container. This test will in most cases provide an erroneous indication of the solids content as would be determined by the dredging industry's method since once the in situ material is distributed it will resettle at a density less than its original density.

16. To avoid any confusion concerning percentage solids, only percent solids by weight will be used to characterize suspended solid concentrations in dredged material slurries.

17. Gradation of Suspended Solids. The grain-size distribution of dredged material is an important parameter which influences the settling processes. Material containing significant amounts of particles with equivalent diameters of a few microns or less may be extremely difficult to settle out effectively, unless natural or induced aggregation of particles occurs. A previous study (Krizek, Fitzpatrick, and Atmatzidis, 1976) investigated the physical properties of fine-grained bottom sediments taken from sixty locations around the country, and the reported range and average values of the grain-size distributions are summarized in Figure 3. These distributions can be arbitrarily divided into zones of coarse, average, and fine material. However, since conventional hydrometer tests require the use of dispersing agents, the information shown does not reflect the naturally occurring agglomeration (and flocculation) of fine particles, particularly in a saline environment. It has been estimated that grain-size distributions of fine-grained material (1 to 10 microns in diameter) in its natural state would show percentages by weight reduced approximately one-half from those illustrated in Figure 3 (Krizek, Fitzpatrick, and Atmatzidis, 1976).

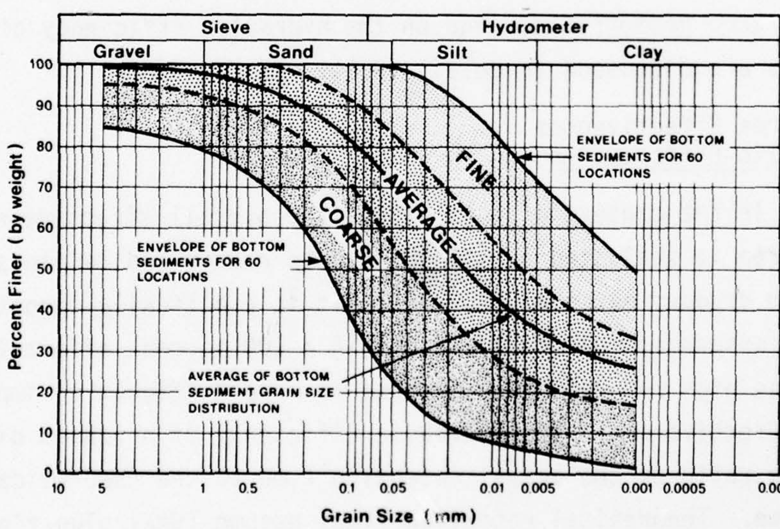


Figure 3. Envelopes of coarse, average, and fine bottom sediment grain-size distributions from sixty locations (After Krizek, Fitzpatrick, and Atmatzidis, 1976)

Geometry of Disposal Areas

18. Ideally, disposal sites should have large surface areas and long retention times. Exclusive of economic considerations, the areas should have a high length-to-width ratio to induce plug flow. This effect can also be obtained by using internal dikes or baffles for sites that are square or irregularly shaped. From a practical viewpoint, most disposal areas conform to the shape of available land. Smaller areas along or near waterways frequently have rectangular shapes, while larger upland areas tend to be irregularly shaped.

19. The most economical disposal areas are very large and square shaped, because the dike length and costs increase proportionally to the square root of the area enclosed. Hence, a 1000-acre site will only cost about 3 to 4 times as much to construct as a 100-acre site. Furthermore, since retention times and settling effectiveness are increased with increasing volume of retained water, the depth of ponded water should be as large as the safety of the surrounding dikes allows. Large open areas of ponded water can be adversely affected by wind, and, therefore, some tradeoffs between area size and shape exist. The effects of area geometry and wind on the hydraulic efficiency of disposal sites are discussed in detail in Part IV.

Disposal Area Effectiveness and Hydraulic Efficiencies

20. In the context of this study, the overall effectiveness of a disposal area is evaluated by its ability to remove and retain suspended solids from dredged material slurries. It is practically impossible to remove all suspended solids and to obtain a 100 percent effectiveness. Furthermore, high removal effectiveness is more difficult to improve than low effectiveness. The hydraulic efficiency of a ponded disposal area is the ratio of the actual retention time to the theoretical retention time. Theoretical retention times assume ideal plug flow conditions, which are practically impossible to obtain. Short-circuiting and inactive areas will decrease retention times and hydraulic efficien-

cies. Although hydraulic efficiencies and removal effectiveness are related, it is possible to have low hydraulic efficiencies and still have high removal effectiveness due to grossly oversized areas. The major objective in the design of an ideally functioning disposal area is to achieve maximum hydraulic efficiency and suspended solids removal effectiveness at minimum cost. The desirable removal effectiveness is imposed according to existing effluent water quality standards.

21. The overall removal effectiveness, E, at any given time, t, of a disposal area is defined in percentage as

$$E = \frac{S_i - S_o}{S_i} \times 100 \qquad (2)$$

where S_i is the concentration of suspended solids in the influent and S_o is the concentration of suspended solids in the effluent of a disposal area. It should be noted that the settling effectiveness is computed as a percentage of removal of the amount of suspended solids in the influent slurry, and this parameter will be used in latter parts of this report, instead of actual slurry or effluent densities, to represent the overall performance of disposal areas.

Effluent Quality Standards

22. The Environmental Protection Agency (EPA) has released interim guidelines for the control of disposal operations (EPA, 1975); these recommend a case-by-case evaluation of the entire disposal operation, including effluent quality, on an ecological impact basis. Accordingly, the U. S. Army Corps of Engineers has prepared a set of interim guidelines for the "Ecological Evaluation of Proposed Discharge of Dredged or Fill Material Into Navigable Waters" (U. S. Army Corps of Engineers, Waterways Experiment Station, 1976). This document describes bioassay procedures, elutriate tests, and other methods for assessing the impact of dredged material discharges in excess of one cubic yard. An important development is the definition of a discharge mixing zone in order

to apply these assay-type tests. These documents state clearly that the definition of dredged material discharges includes the runoff or overflow from a contained land or water disposal. Therefore, any discharge from dredged material containment areas must be evaluated for potential environmental impacts, regardless of the amount of suspended solids remaining in the effluents. Although necessary for ecological impact evaluations, the subject guidelines do not propose any specific limitations on the concentration of suspended solids in the effluents of a disposal area and, therefore, are not useful from a design viewpoint.

23. Previous studies of disposal operations (Murphy and Zeigler, 1974; Krizek, Fitzpatrick, and Atmatzidis, 1976) have indicated that no uniform or national guidelines exist for controlling disposal area effluent quality. Several types of effluent quality standards, each based on a different parameter, are imposed by local or State guidelines and are used by Corps of Engineers District offices. The most commonly used standard is limiting the amount of suspended solids in the effluent to between 4 and 13 g/l above the ambient water levels. Other parameters used are turbidity units (JTU) and Settleable Solids (ml/l). In Table 1 (Krizek, Fitzpatrick, and Atmatzidis, 1976) are summarized the standards which were in use during 1975, and no significant changes have occurred to date (1977), with the exception that a revised criterion of 50 JTU is now in effect in the Portland District. During the site visits conducted as part of this study, it became apparent that tighter controls and stricter standards are likely to be imposed on disposal area effluents in the future and that these standards should be made uniform and as simple as possible for effective planning and inspection of operational performance.

Conclusions

24. On the basis of the information presented in this Part, the following conclusions can be advanced:

Table 1

Water Quality Standards for Disposal Area Effluents+

District	1975 Standard
Galveston	8 g/l above ambient
New Orleans	1.5 x ambient concentration
Mobile	50 JTU above ambient
Jacksonville	50 JTU above ambient
Savannah	-
Charleston	-
Wilmington	50 JTU above ambient
Norfolk	13 g/l above ambient
Philadelphia	8 g/l above ambient*
	4 g/l above ambient**
New York	1.5 x ambient concentration
Buffalo	None set
Detroit	No standards
Chicago	None set
Sacramento	6 g/l above ambient
Portland	1.5 x ambient concentration
Seattle	5 JTU (state requirement)
	5 g/l above ambient (Corps criterion)
Los Angeles	None set
San Francisco	None set

* Small-size areas ** Large-size areas

\+ Standards were State imposed or voluntarily imposed by the Districts in cases where no State standards existed

a. Conventional hydrometer tests with dispersing agents do not reflect naturally occurring agglomeration of fine particles. This can result in distorted grain-size distributions which show higher percentages of fine material than effectively exist.

b. The removal effectiveness (percentage of retained suspended solids) reflects the overall performance of a disposal area and is a convenient parameter to use for evaluating the design and operation of sedimentation basins.

c. The hydraulic efficiency of a dredged material sedimentation basin is the ratio of actual retention time to ideal retention time and is an important parameter in the design and operation of a disposal area. Both the hydraulic efficiency and effective area of a sedimentation basin should be as high as possible in order to achieve maximum settling effectiveness at minimum cost.

d. No simple test or guidelines exist to evaluate the environmental impact of effluents discharged from dredged material containment areas. The most commonly used standard in the past for controlling disposal area effluent quality limits the amount of suspended solids in the effluent to between 4 and 13 g/l above ambient water levels.

PART III: SITE VISITS AND FIELD SAMPLING

Site Visits

25. Ten Corps of Engineers Districts involved in active disposal operations were visited during this research project. The purpose of these visits was to obtain a practical understanding of the problems encountered in disposal operations and to collect the most recent information available on disposal practices. Samples of disposal area influents and effluents were collected at a variety of sites which ranged in size from a few acres to 2000 acres. Other records and data were obtained and discussions were held with District personnel to gain insight into the guidance requirements for future disposal operations. The Districts visited represent operations occurring in all sections of the country where active confined disposal programs are being administered by the Corps of Engineers. It is believed that the ten-District sample provides a fair representation of the general problems associated with confined disposal; however, every District has certain needs and problems that are unique to its circumstances. The detailed narrative reports of information obtained from the site visits are presented in Appendix A and provide a cross section of case studies from each District. This information is qualitatively summarized below in the form of an outline to provide a brief overview of recent disposal operations.

Philadelphia

Water Environment:	Fresh
Type of Material:	Fine silts and sand
Type of Dredging:	Hopper, large-diameter pipes
Disposal Areas:	Large, compartmentalized, heavily vegetated
	Located near shoaling areas along rivers
	Shapes conform to the lay of land
	Medium to high dikes undergoing raising
	Designed for storage and low ponding heights
	Considerable use of cross dikes and spur dikes
Weir Designs:	Large, steel frames perpendicular to dikes
Operating Policies:	Keep ponding depth low, get water out quickly
Design Guidelines:	Very few, mostly based on experience
Effluent Guidelines:	8 g/l suspended solids above ambient
Disposal Costs:	About 20% of total costs or \$0.15 to \$0.50/cu yd
Major Problems:	Dike seepage, weed fires in dry disposal areas
Future Needs:	Better guidelines, possibly long-range disposal
Storage Available:	About 20 years with present sites

Mobile

Water Environment:	Saline
Type of Material:	Silts, silty clays, and sand
Type of Dredging:	Hydraulic pipeline, large-diameter pipes
Disposal Areas:	Large rectangular shapes, some vegetation
	Centrally located on island in harbor
	Moderate height dikes with plastic liners
Weir Designs:	Rectangular steel boxes 8 ft x 4 ft in dikes
Operating Policies:	Moderate ponding depths
Design Guidelines:	Very few, use space and material available
Effluent Guidelines:	Suspended solids and turbidity (Florida)
Disposal Costs:	Included in dredging costs of $0.81/cu yd
Major Problems:	Obtaining adequate space, contractual limitations making site management difficult
Future Needs:	Guidelines on new site design and on better management of existing sites

Galveston

Water Environment:	Saline
Type of Material:	Silty clays
Type of Dredging:	Hydraulic pipeline, large-diameter pipes
Disposal Areas:	Variable sizes, small to large
	Located near dredging areas for convenience
	Generally very flat and open, little vegetation
	Frequently compartmentalized or built with one spur dike near sluice
Weir Designs:	Large drop inlets about 30 ft in from dikes
Operating Policies:	Moderate ponding depth based on effluent quality
Design Guidelines:	Very few
Disposal Costs:	Included in dredging costs
Major Problems:	Obtaining space, wind effects, and resuspension
Future Needs:	Better site design and management guidelines

Portland

Water Environment:	Saline, brackish, fresh
Type of Material:	All types
Type of Dredging:	Hydraulic pipeline, small to large diameters
Disposal Areas:	Extremely diverse, very small to large sites
	Frequently compartmentalized with berms
	Various areas from sand dunes to pasture lands
	Large sites located far from dredging areas requiring long-distance pumping
Weir Designs:	Small culverts to very large steel frames
Operating Policies:	Moderate ponding based on effluent quality
Design Guidelines:	Very few, design for fishery protection

Portland (Continued)

Effluent Guidelines:	Jackson turbidity units, mostly 50 JTU's
Disposal Costs:	15% to 20% of dredging costs
Major Problems:	Obtaining adequate space closeby
Future Needs:	Better site design and management guidelines based on type of material and dredge

Seattle

Water Environment:	Saline, brackish, fresh
Type of Material:	Coarse to fine, some wood-type organics
Type of Dredging:	Hydraulic pipeline, medium to large diameters
Disposal Areas:	Very diverse with many sizes and shapes Areas planned by several involved groups Located close to dredging on variety of sites Some sites operated in series
Weir Designs:	Large variety, small to very large drop inlets Fixed horizontal boards in front of sluices
Operating Policies:	Protect fishery resources
Design Guidelines:	Very few, maintain low weir crest heights
Effluent Guidelines:	50 ml/l settleable solids, turbidity, and many other bioassay test requirements
Disposal Costs:	Very variable dependent on circumstances
Major Problems:	Meeting very difficult effluent standards
Future Needs:	Must have uniform and reliable design guidelines to meet specific standards

Norfolk

Water Environment:	Saline, brackish, fresh
Type of Material:	Much sand and silty shoals
Type of Dredging:	All types including large-diameter pipes
Disposal Areas:	Very large "in-water" structures and other small to moderate size areas Most areas heavily vegetated Some multi-leveled compartmental sites Extensive use of finger dikes and berms
Weir Designs:	Large E-shaped weirs with very large total weir lengths and prefabricated steel drop inlets
Operating Policies:	Try to induce filtering through vegetated areas
Design Guidelines:	Very little, based on experience and economics 14-ft weir length for pipe under 12 inches, 28-ft weir lengths for larger pipes
Effluent Guidelines:	13 g/l suspended solids above ambient
Disposal Costs:	\$0.07/cu yd (large areas) to \$0.34/cu yd (small areas)

Norfolk (Continued)

Major Problems:	Control of rain water and protection of dikes
Future Needs:	Obtaining space and better design guidelines Resolve ultimate disposition of filled dikes

Baltimore

Water Environment:	Saline, brackish
Type of Material:	Sand, and fine silt
Type of Dredging:	Hydraulic pipeline and hopper; 12-to 16-inch pipes
Disposal Areas:	Mostly moderate size near dredging areas All material is confined on land Considerable use of compartmentalized and finger dikes Some sites have two disposal areas for separate operation or compartmentalization Area size based on type of material
Weir Designs:	Three-sided concrete walls, 12-feet total length
Operating Policies:	Contractor has considerable freedom as long as effluent quality is maintained
Design Guidelines:	Maximum dike height is 10 ft
Effluent Guidelines:	13 g/l suspended solids above ambient
Disposal Costs:	Included in dredging costs averaging $1.35/cu yd
Major Problems:	Obtaining suitable sites from local interests
Future Needs:	Improved guidelines for disposal site design

Charleston

Water Environment:	Saline, brackish, fresh
Type of Material:	Silty clays
Type of Dredging:	Hydraulic pipeline, medium to large diameters
Disposal Areas:	Many rectangular sizes acquired by easements along waterways (1000 ft wide) Several very large irregularly shaped areas Located in low-lying marshes; heavily vegetated Low dikes used and later built up
Weir Designs:	Standard 6-ft-wide Armco-type arranged in pairs
Operating Policies:	Redistribute material, keep water levels low
Design Guidelines:	Very few, work with sites available
Effluent Guidelines:	Monitored visually
Disposal Costs:	Previously $0.02 to $0.06/cu yd, now estimated to be $0.04 to $0.12/cu yd
Major Problems:	Redistribution of material, some channelization
Future Needs:	Improved guidelines for sizing and design of disposal areas
Storage Available:	Quite a few years

Savannah

Water Environment:	Saline, brackish
Type of Material:	Various types
Type of Dredging:	Hydraulic pipeline, large-diameter pipes
Disposal Areas:	Very large, open areas but heavily vegetated Multiple inlets and outlets for convenience No internal dikes or compartmentalization Moderate dikes raised high by lifting programs Some channelled areas to redistribute material
Weir Designs:	6-ft-wide risers now made of aluminum
Operating Policies:	Keep areas dry, use filtering of vegetation
Design Guidelines:	Very few, get as much area as possible
Effluent Guidelines:	Visually monitored, some have turbidity limits
Disposal Costs:	Dikes constructed for $0.75 to $1.00/cu yd of dike material Weirs cost $2000 each with sluice pipes
Major Problems:	Dike failures; release of heavy rain runoff
Future Needs:	New guidelines for improved disposal and management practices
Storage Available:	About 10 years

Vicksburg

Water Environment:	Fresh
Type of Material:	Silts, clays, and colloidal particles
Type of Dredging:	Hydraulic pipeline, medium-size pipes
Disposal Areas:	Uniform design of long and narrow shapes with two separate compartments Presently small areas located along rivers, expect larger ones in future High dikes used for deep ponding
Weir Designs:	Usually 100-ft-wide internal weirs with drop inlet sluice in final pond
Operating Policies:	Maintain maximum retention; use deep ponding
Design Guidelines:	Developing new guidelines based on field tests, usually maintain 2-inch crest heights
Effluent Standards:	Suspended solids and visual monitoring
Disposal Costs:	Up to a $1.00/cu yd for dike construction Unit costs per acre are high due to small sites
Major Problems:	Difficult to settle colloidal particles Efficiency is high, but effluents still turbid
Future Needs:	Proper design and construction of new large areas required for future projects

26. It was observed that available guidance for the management of existing sites or for the proper planning and design of new sites is insufficient. The guidelines most frequently in use include (a) maintaining low water heads over the weir crest and (b) limiting the concentration of suspended solids in the effluent to about 4 to 13 g/l above ambient water concentrations. Policies on ponding depths vary widely and depend upon local considerations. Costs of disposal areas also vary considerably but appear to average about 15 percent to 20 percent of the total dredging costs. It is believed that strict controls will be imposed on disposal operations in the near future; and new, uniform, and comprehensive guidelines are required for the proper design, operation, and maintenance of disposal areas.

Field Samples

27. Forty-four samples of influent slurries, supernatants, and effluents were collected from nine active disposal areas during site visits. The concentration of suspended solids for all samples and the gradation of suspended solids in influent slurries were determined. A list of all samples, including descriptions and suspended solids concentrations, is given in Table 2. Gradations determined with and without the use of a dispersing agent are presented in Figures 4-6. It can be observed that the use of a dispersing agent substantially distorts the natural gradation of dredged bottom sediments. For the materials shown, the amount of grains finer than 1 micron ranges from 20 percent to 80 percent when a dispersing agent is used, while the ranges observed without the use of a dispersing agent are 5 percent to 60 percent, for those samples tested by both methods.

28. Influent slurry densities varied between 60 g/l (5.8 percent by weight) and 317 g/l (26.5 percent by weight) with an average value of 172 g/l (15.6 percent by weight). Suspended solids in the effluents ranged from a low of 0.03 g/l to a high of 15.7 g/l.

Table 2

Field Samples

Sample	Date	Location	Disposal Site	Description	Suspended Solids g/l	Suspended Solids % weight
MA1D	9-23-76	Mobile, Ala.	South Blakeley Island	Discharge into disposal area	60.37	5.821
MA2D	9-23-76	Mobile, Ala.	South Blakeley Island	Discharge into disposal area	131.84	12.195
MA3W	9-23-76	Mobile, Ala.	South Blakeley Island	Weir overflow	0.207	0.021
MA4W	9-23-76	Mobile, Ala.	South Blakeley Island	Weir overflow	0.150	0.015
MA5E	9-23-76	Mobile, Ala.	South Blakeley Island	Effluent from sluice pipe	0.186	0.019
MA6E	9-23-76	Mobile, Ala.	South Blakeley Island	Effluent from sluice pipe	0.179	0.018
MA7E	9-23-76	Mobile, Ala.	South Blakeley Island	Effluent from sluice pipe	0.271	0.027
FT1M	9-24-76	Freeport,Tex.	Area No. 3	Mud residual of inactive site	-	-
FT2S	9-24-76	Freeport,Tex.	Area No. 2	Supernatant of disposal area midpoint	20.22	2.00
FT3W	9-24-76	Freeport,Tex.	Area No. 2	Weir overflow (clear layer)	0.429	0.043
FT4W	9-24-76	Freeport,Tex.	Area No. 2	Weir overflow (turbid layer)	15.69	1.554
FT5E	9-24-76	Freeport,Tex.	Area No. 2	Effluent from sluice pipe	15.38	1.524
FT6D	9-24-76	Freeport,Tex.	Area No. 2	Discharge into disposal area	317.04	26.531
IW1D	10-26-76	Ilwaco, Wash.	Ilwaco Harbor	Discharge into disposal area	311.5	26.142
IW2W	10-26-76	Ilwaco, Wash.	Ilwaco Harbor	Weir overflow of internal weir	0.193	0.019
IW3W	10-26-76	Ilwaco, Wash.	Ilwaco Harbor	Weir overflow of internal weir	0.093	0.009
IW4E	10-26-76	Ilwaco, Wash.	Ilwaco Harbor	Effluent from sluice pipe	0.029	0.003
WW1W	10-27-76	Willapa,Wash.	No. 2 (active)	Weir overflow of internal weir	0.086	0.008
WW2S	10-27-76	Willapa,Wash.	No. 2 (active)	Supernatant of second compartment	36.0	3.522
WW3E	10-27-76	Willapa,Wash.	No. 2 (active)	Effluent from sluice pipe	0.243	0.024
WW4W	10-27-76	Willapa,Wash.	No. 1 (inactive)	Weir overflow of rain water	0.036	0.004
WW5D	10-27-76	Willapa,Wash.	No. 2 (active)	Discharge into disposal area	147.6	13.532
WW6S	10-27-76	Willapa,Wash.	No. 2 (active)	Supernatants below influent pipe	99.6	9.385

Table 2 (Continued)

Sample	Date	Location	Disposal Site	Description	Suspended Solids g/l	Suspended Solids % weight
NV1M	10-76	Norfolk, Va.	Craney Island	Mud residual of recent sediments	-	-
NV2M	10-76	Norfolk, Va.	Craney Island	Mud residual of recent sediments	-	-
NV3M	10-76	Norfolk, Va.	Craney Island	Mud residual of recent sediments	-	-
NV4M	10-76	Norfolk, Va.	Craney Island	Mud residual of recent sediments	-	-
CC1D	12-3-76	Charleston, S.C.	Yellow House Creek	Discharge into disposal area	147.08	13.488
CC2W	12-3-76	Charleston, S.C.	Yellow House Creek	Weir overflow of 1st weir	9.94	0.988
CC3E	12-3-76	Charleston, S.C.	Yellow House Creek	Effluent of 1st sluice pipe	6.47	0.644
CC4W	12-3-76	Charleston, S.C.	Yellow House Creek	Weir overflow of 2nd weir	1.21	0.121
CC5E	12-3-76	Charleston, C.C.	Yellow House Creek	Effluent of 2nd sluice pipe	7.35	0.732
SG1E	12-1-76	Savannah, Ga.	Area No. 13	Effluent from disposal area	0.21	0.021
SG2E	12-16-76	Savannah, Ga.	Area No. 13	Effluent from disposal area	0.521	0.052
SG3E	12-22-76	Savannah, Ga.	Area No. 13	Effluent from disposal area	0.200	0.020
SG4E	3-1-77	Savannah, Ga.	Area No. 13	Effluent from disposal area	0.450	0.045
SG5E	3-2-77	Savannah, Ga.	Area No. 13	Effluent from disposal area	7.186	0.715
BG1M	12-2-76	Brunswick, Ga.	Andrews Island	Mud residual of recent sediments	-	-
BG2M	12-2-76	Brunswick, Ga.	Andrews Island	Mud residual of recent sediments	-	-
BG3E	1-6-77	Brunswick, Ga.	Andrews Island	Effluent from disposal area	0.271	0.027
BG4E	1-8-77	Brunswick, Ga.	Andrews Island	Effluent from disposal area	0.279	0.028
YM1D	2-10-77	Yazoo City,Miss.	Area No. 6	Discharge into disposal area	158.54	14.446
YM2W	2-23-77	Yazoo City,Miss.	Area No. 5	Weir overflow of 1st weir	2.24	0.224
YM3B	2-23-77	Yazoo City,Miss.	Area No. 5	Bottom sample 3-5' below weir	12.27	1.218

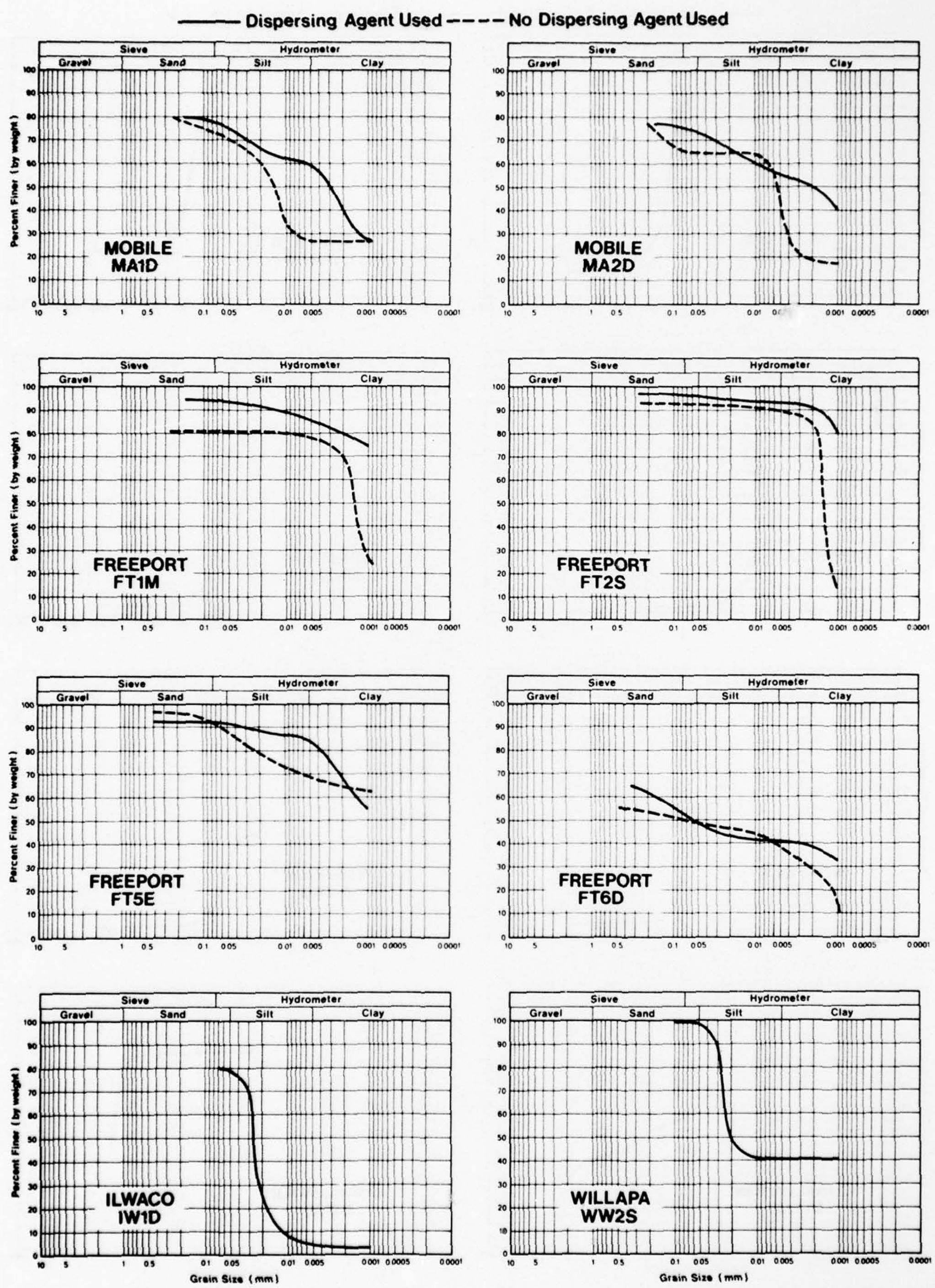


Figure 4. Grain-size distributions of field samples from the Mobile, Freeport, Ilwaco, and Willapa disposal areas

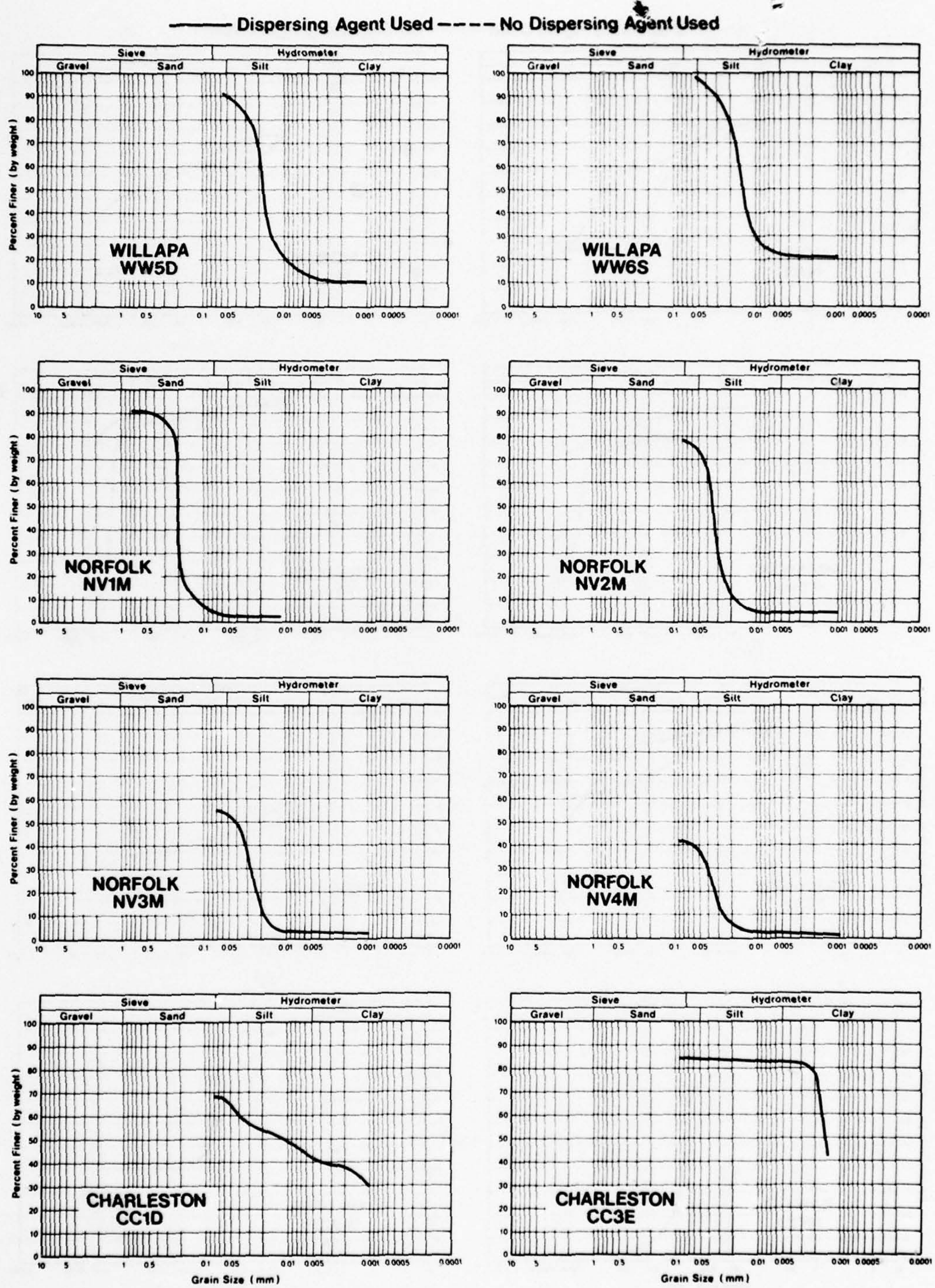


Figure 5. Grain-size distributions of field samples from the Willapa, Norfolk, and Charleston disposal areas

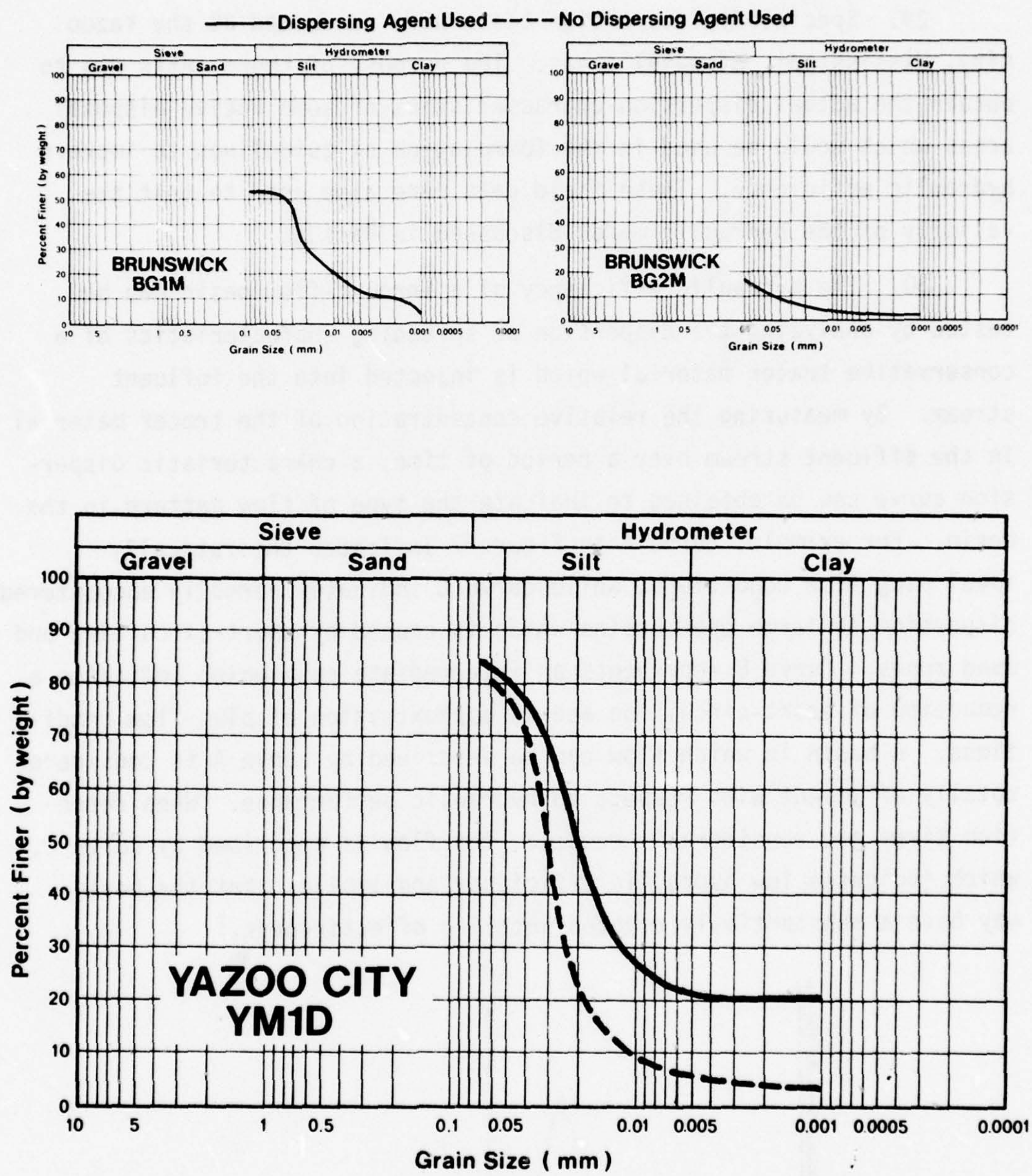


Figure 6. Grain-size distributions of field samples from the Brunswick and Yazoo City disposal areas

Dye-Dispersion Tests

29. Special dye-dispersion tests were conducted at the Yazoo City, Mississippi, disposal areas. The purpose of these tests was to obtain the actual dispersion characteristics of some active disposal areas which would be used in the formulation of guidelines to improve hydraulic efficiency. These field data were also used to test the validity of the hydraulic model discussed in Part IV.

30. The hydraulic efficiency of a through-flow basin can be tested by analyzing the dispersion or spreading characteristics of a conservative tracer material which is injected into the influent stream. By measuring the relative concentration of the tracer material in the effluent stream over a period of time, a characteristic dispersion curve can be obtained to indicate the type of flow pattern in the basin. For example, curve A in Figure 7 indicates theoretically ideal plug flow conditions, while curve C indicates normally encountered dispersion in large open basins which is caused by short-circuiting and dead zones. Curve B represents an intermediate case which indicates a reduction of short-circuiting and an approximation of plug flow conditions. A basin in which flow can be described by curve A is considered totally efficient with respect to hydraulic performance. When retention times are considerably reduced, the flow is described by curve C, which indicates low hydraulic efficiency and implies that the basin may have a substantially reduced settling effectiveness.

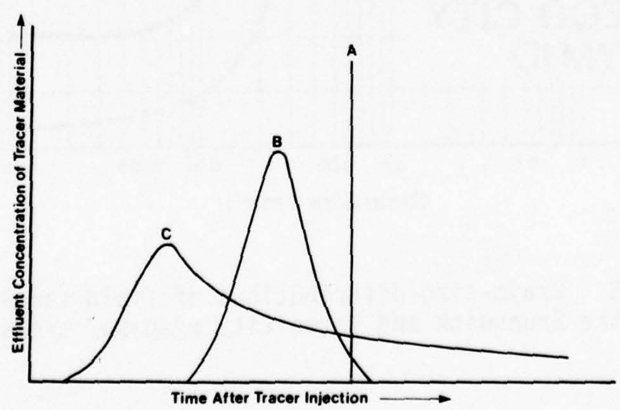


Figure 7. Typical dispersion curves

31. The dispersion tests were conducted at disposal areas 5 and 6 near Yazoo City, Mississippi, using a fluorescent dye (Rhodamine WT). This dye re-emits light of a particular wavelength when excited by some light source. Approximately one gallon of 20 percent dye solution was injected into the slurry discharge stream and samples of the first weir effluents were tested with a Turner Model 111 Fluorometer. This device can detect very minute traces of dye in parts per billion by a sensitive photo tube and optical filter system especially designed to block out interferences. The excitation lamp used was a Far UV lamp; the primary filter was a 546-nm sharp cutoff filter; and a 590-nm narrow bandpass filter was used as the secondary filter. A high-volume flow-through measuring device and pumping system was used (Figure 8) to provide continuous monitoring ability. This arrangement worked quite well and provided very good sensitivity and selectivity.

32. Diagrams of the two disposal areas and the results of all dye-dispersion tests are shown in Figures 9 and 10. It can be observed that strong dispersion was measured at these sites. Actual retention times were substantially less than ideal retention times, which correspond to plug flow and are equal to the basin volume divided by the average discharge rate (V_B/Q). The actual retention times were further reduced when wind blew over the disposal area, as shown in Figure 9. The effect of the ponded water volume on the retention times is clearly illustrated by the dispersion curves obtained in area 5 for ponding depths of approximately 8 ft and 2 ft (Figure 10). Although similar dispersion effects were obtained in both cases, the retention times were much shorter for the smaller ponding depth. Due to the decreased retention times and the increased possibility of bottom sediment resuspension, the suspended solids removal effectiveness of the basin had deteriorated substantially with the smaller ponding depth.

33. On the basis of these results, it can be concluded that considerable short-circuiting occurs in large shallow basins, and this results in inefficient flow patterns. Actual retention times are significantly shorter than ideal through-flow times, and wind effects can be

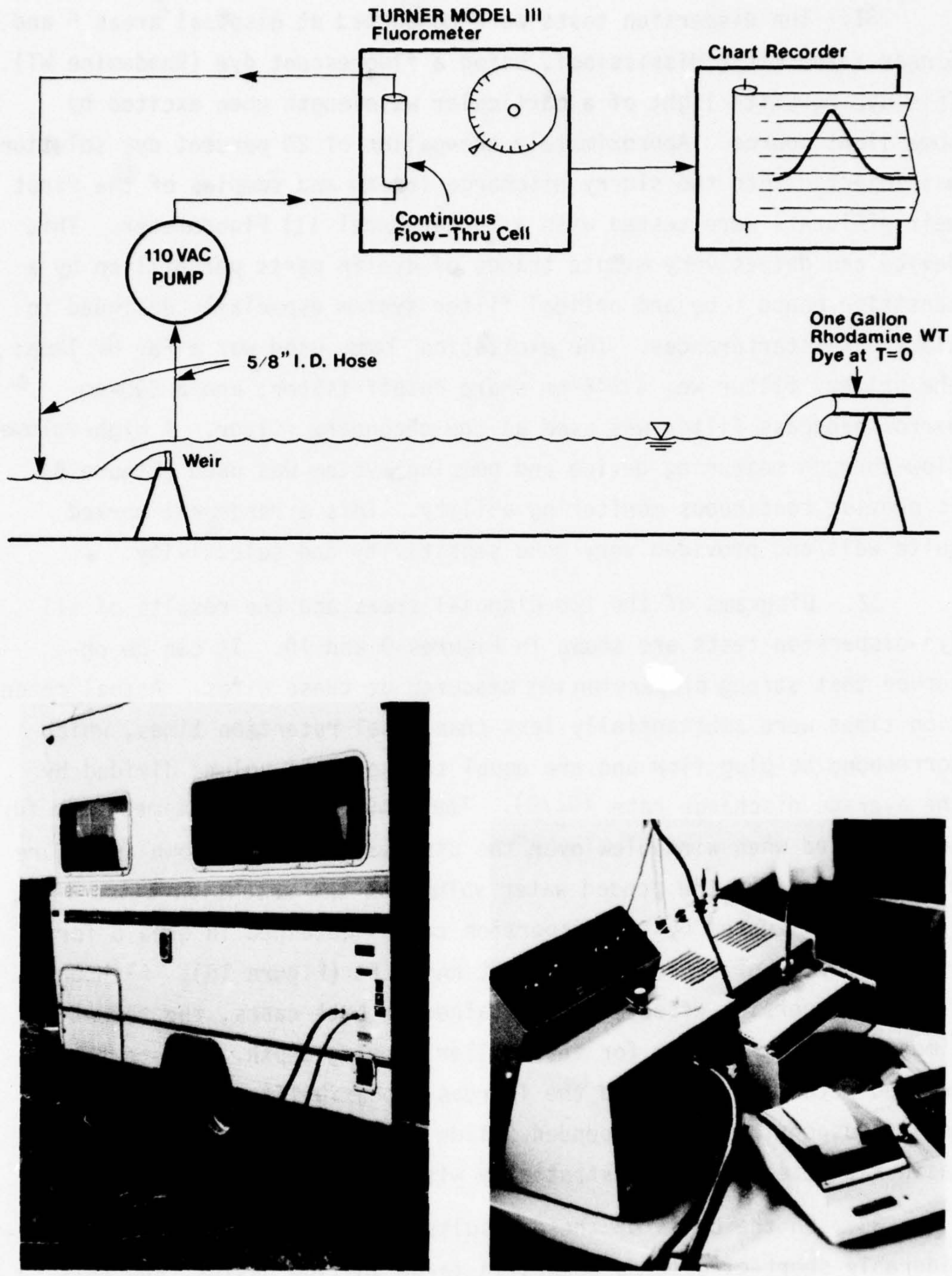
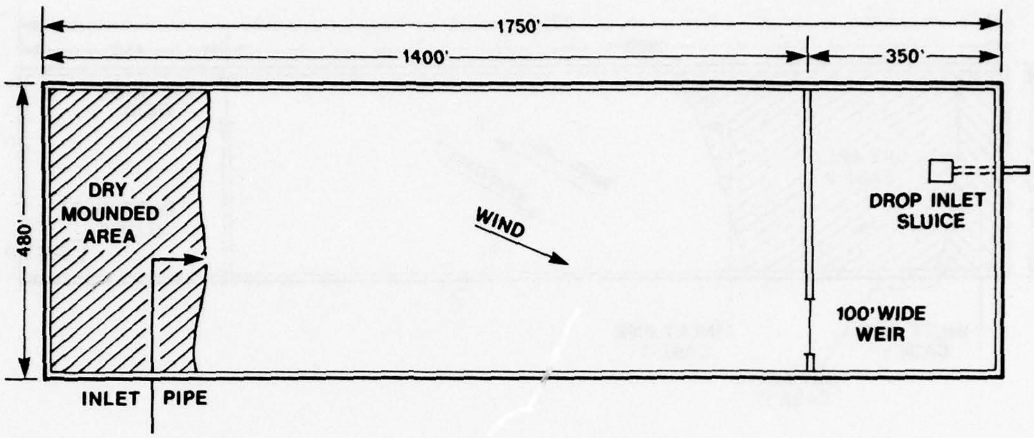
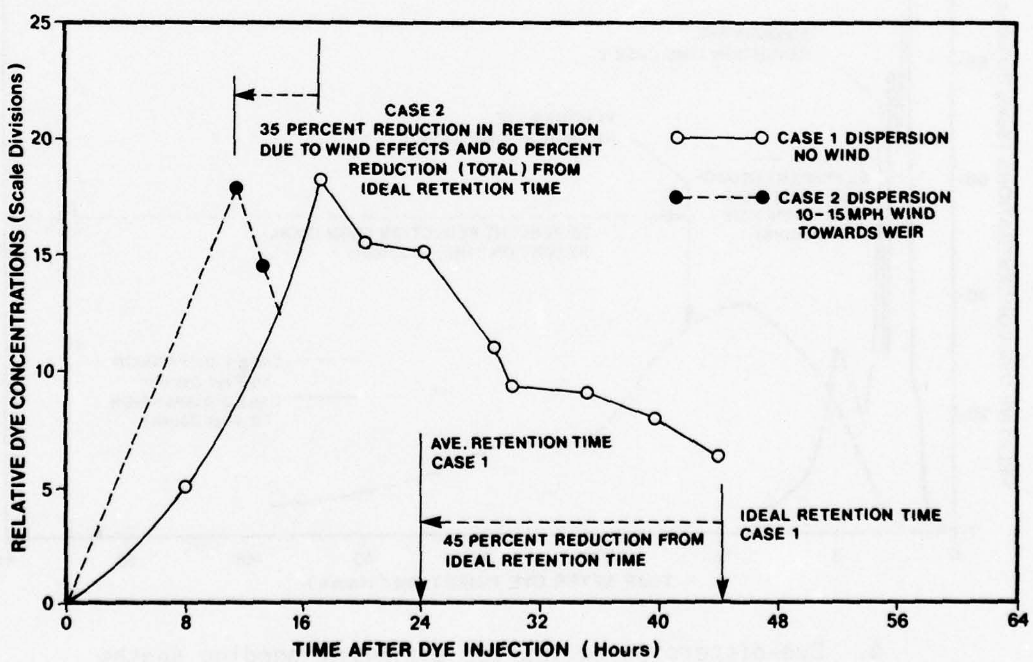


Figure 8. Equipment and setup used for dye-dispersion tests

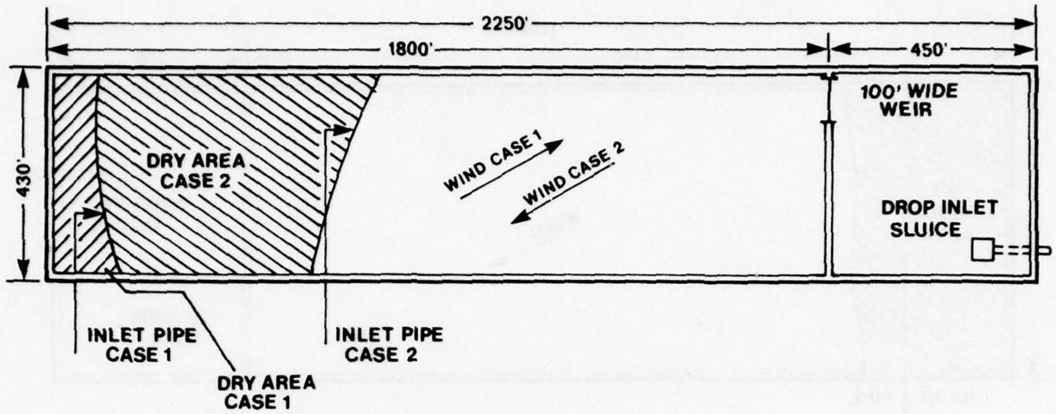


a. Plan of disposal area 6

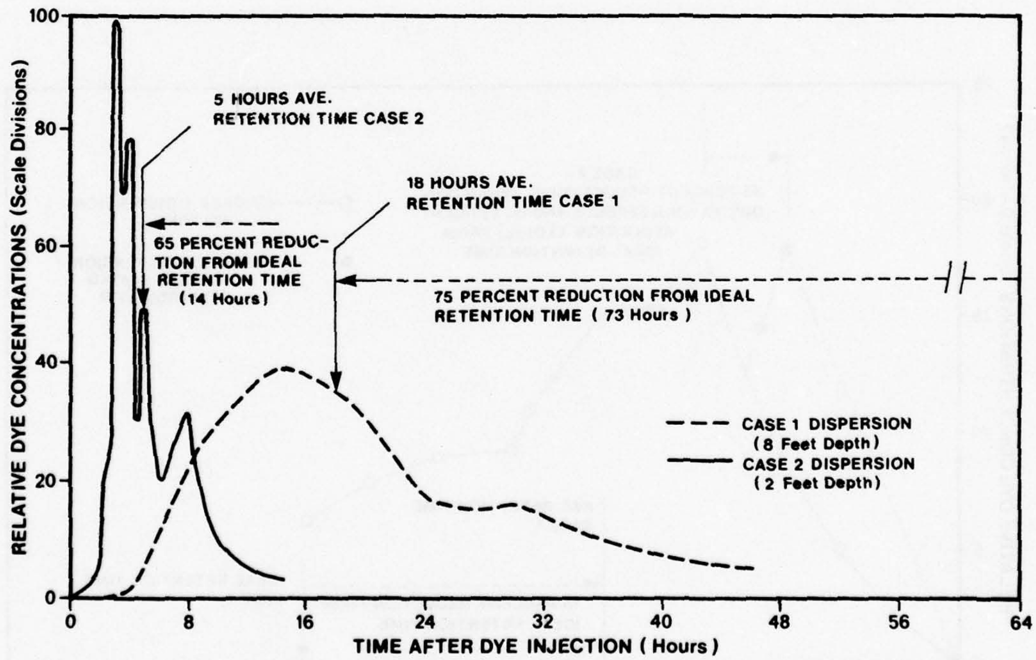


b. Dye-dispersion curves for different wind conditions

Figure 9. Dye-dispersion test conditions and results for disposal area 6, Yazoo City, Mississippi



a. Plan of disposal area 5



b. Dye-dispersion curves for different ponding depths

Figure 10. Dye-dispersion test conditions and results for disposal area 5, Yazoo City, Mississippi (Case 2 data courtesy of R. Montgomery)

very detrimental. Furthermore, increasing ponding depth increases the retention time and reduces the possibility of bottom sediment resuspension due to high through-flow velocities or wind-induced currents.

Qualitative Analysis of Effectiveness

34. Samples of influent slurries and effluent waters were obtained from eight active disposal areas (Table 3). The suspended solids concentrations of these samples were used to determine the removal effectiveness of each basin. The computed effectiveness values, together with pertinent information on the size, ponding depth, and flow rate of each area, are summarized in Table 3. It can be observed that good to excellent suspended solids removal was achieved at these sites, ranging from 88 percent to 99.9 percent with a remarkably high average of about 96 percent. In subsequent parts of this report, it will be shown that the shape of the basin, the induced flow pattern, and the method of releasing supernatants have an important effect on the performance of a disposal area. A qualitative evaluation of disposal area effectiveness will be presented next on the basis of field observations and data. Experience indicates that the effective area, the ponding depth, and the influent flow rate are three important factors affecting the performance of a disposal area. Although performance is also affected by the characteristics of the influent slurry, these parameters will not be considered in this simplified evaluation.

35. To quantify the ideal plug-flow retention time of a disposal area, the concept of a retention factor, RF, is introduced. This factor is defined as

$$RF = \frac{A_e \times D_p}{Q_i} \tag{3}$$

where the retention factor, RF, is in 12-hour (½-day) units; the effective settling area, A_e, is in acres; the average ponding depth, D_p, is in feet; and the average influent flow rate, Q_i, is in cfs.

Table 3

Performance of Disposal Sites

Disposal Site	Effective Area (acres)	Ponding Depth (ft)	Inflow Rate (cfs)	Retention Factor (½ days)	Removal Effectiveness (%)
Ilwaco, Wash.	10	6	6	10.0	99.9
Mobile, Ala.	275	2	60	9.2	99.8
Savannah, Ga.	1000	0.5	60	8.3	98.8
Charleston, S. C.	250	1	30	8.3	96.0
Yazoo City, Miss. (1)	15	8	18	6.7	98.6
Freeport, Tex.	45	3	45	3.0	95.0
Yazoo City, Miss. (2)	11	2	18	1.2	93.0
Willapa, Wash.	12	3	32	1.1	88.0

Due to the convenient units selected, the retention factor will be a number usually smaller than 10, which can be used as a performance indicator. For example, a disposal site with an effective area of 100 acres, an average ponding depth of 2 feet, and a flow rate of 40 cfs (24-inch pipeline) would have a retention factor of 5. The retention factors of the eight disposal areas that were sampled during site visits are shown in Table 3 and are plotted versus the corresponding effectiveness value, E, in Figure 11. When not accurately known, effective areas were estimated as a fraction of the total disposal area size (Part VII of the report provides a method for estimating effective areas of containment basins).

36. It can be observed that larger retention factors correlate with increased effectiveness of a disposal area with respect to solids removal. The dashed line in Figure 11 represents an arbitrary function of the retention factor to illustrate this correlation. High retention factors may not always ensure the effectiveness of disposal areas. Therefore, under certain conditions, areas with higher retention factors could have lower removal effectiveness. One example is the Charleston disposal area, which had a higher retention factor but a lower effectiveness than Yazoo City (case 1). However, reference to Figures 5 and 6 for samples CC1D and YM1D shows that Charleston had slurries characterized as "fine grained" while Yazoo City slurries were "average grained", as based on data shown in Figure 3. This could account for the greater settling effectiveness obtained at Yazoo City with a lower retention factor. It must be cautioned that the correlation suggested between retention factors and removal effectiveness is based on limited data and must be considered qualitative. Much more data, particularly accurate material characterizations, are required to refine this concept. Estimates of effective areas and ponding depths can introduce significant errors; and better procedures to determine these parameters are required. Nevertheless, this simple concept is useful for demonstrating how increased retention times maximize hydraulic efficiencies and improve settling effectiveness.

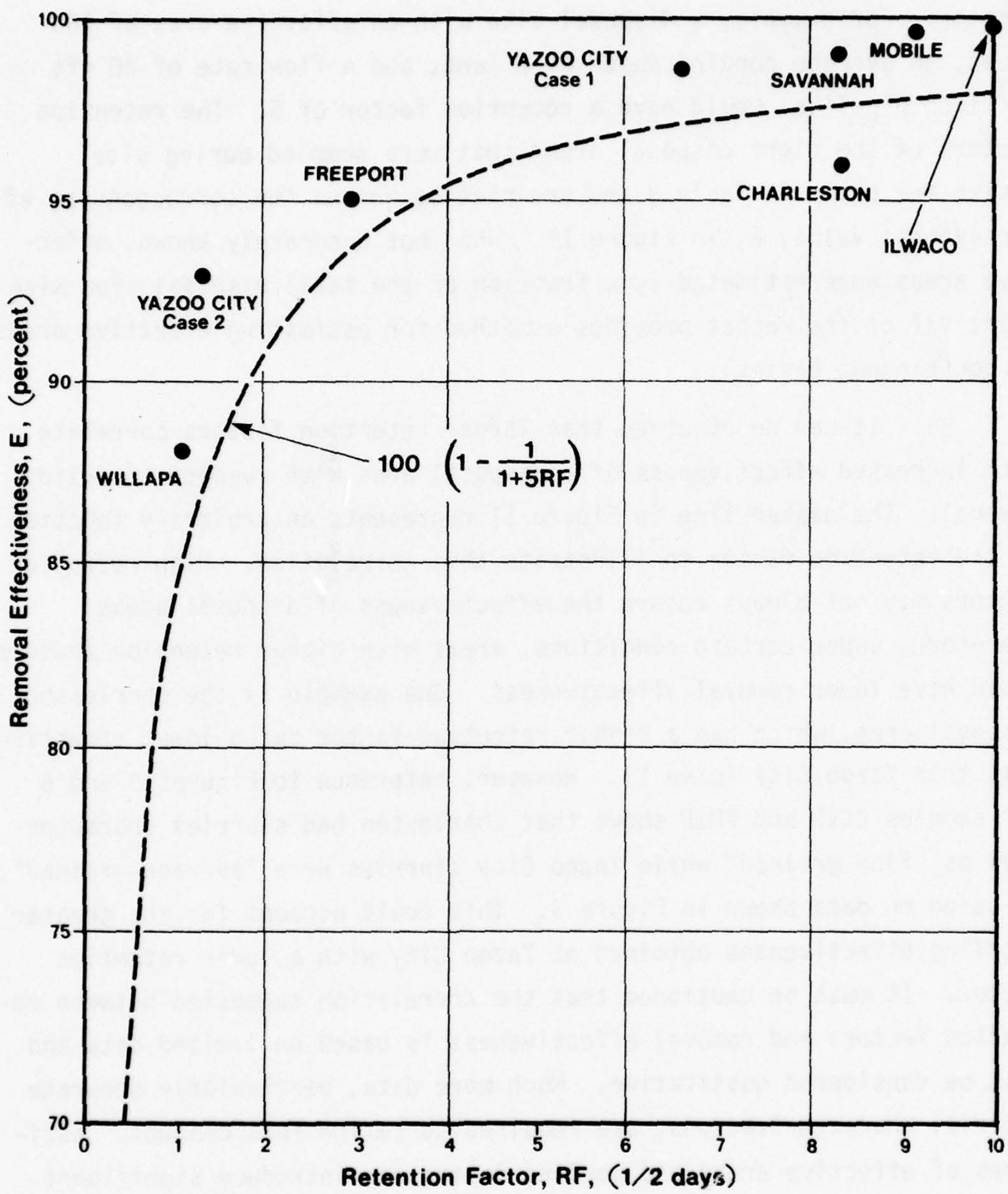


Figure 11. Correlation of disposal site performance data with retention factors

Conclusions

37. On the basis of the foregoing information and discussions, the following conclusions can be advanced.

- a. Most Corps of Engineers Districts have experienced problems in dredged material containment operations, and guidelines for the proper sizing, design, and operation of a disposal area are very limited.
- b. Good disposal area performance is frequently realized, but a substantial improvement in solids retention capability is very difficult to achieve with present design and operation practices.
- c. Poor disposal area performance is usually due to insufficient retention time and/or inadequate management of the facilities (such as improper adjustment of weir crest elevation).
- d. In many cases high settling effectiveness is achieved by use of oversized areas.
- e. The hydraulic efficiencies evaluated for two typical disposal areas were low due to short-circuiting and wind effects.
- f. There is a strong correlation between retention time (hydraulic efficiency) and suspended solids removal; therefore, improving the hydraulic efficiency improves the settling effectiveness by increasing the retention time.
- g. Larger ponding depths increase the effective area and volume of a disposal site and, therefore, increase the retention time and the settling effectiveness.

PART IV: INFLUENCE OF BASIN HYDRAULICS

38. The hydraulic efficiency of a dredged material containment area is directly influenced by the prevailing flow pattern. Water movement in a disposal area is generated by the through-flow from the discharge pipe to the outlet weir and by wind blowing over the surface of the basin. The proper design of a disposal area requires a knowledge of (a) the flow pattern (streamlines or fluid particle trajectories) between inflow and outflow points, (b) the effects of wind on the flow and velocity fields, and (c) the distribution of retention times. An exact mathematical solution for the flow field in a disposal area cannot be obtained because the flow is turbulent. However, a theoretical approach was developed (Appendix B) to predict the flow field in a homogeneous shallow basin with through-flow and superimposed wind. Justifications of the applicability and detailed mathematical derivations are presented in Appendix B. The important physical parameters incorporated in the theoretical development are through-flow, wind stress, eddy viscosity, bottom topography, and disposal area geometry. A general solution for the transport stream function was obtained for a rectangular basin of constant depth with uniform wind and arbitrary locations of the inflow pipe and outflow weir. Then, solutions for particular cases were obtained, and methods were also advanced for determining the effects of wind on the velocity field and the distribution of retention times.

Flow Patterns in Model Basins

39. The flow pattern in a disposal area may be represented by the transport stream function, ψ, which is defined as

$$\frac{\partial\psi}{\partial y} = \int_{-h}^{0} u\,dz \tag{4}$$

and

$$\frac{\partial\psi}{\partial x} = -\int_{-h}^{0} v\,dz \tag{5}$$

where x and y are the horizontal coordinates; z is the vertical coordinate measured upward from the free surface; h is the depth of the water in the disposal area; and u and v are the velocity components in the x and y directions, respectively. The physical meaning of the transport stream function is the same as that of the two-dimensional stream function commonly used in fluid mechanics with the exception that the dimensions of the former are in volume per time. For steady flow, the streamlines coincide with the trajectories of the fluid particles. A uniform wind does not alter the transport stream function in a basin with constant depth, because wind generates circulation at any point in the basin and bottom return flow compensates forward flow in the upper layers.

40. The theoretical approach for determining the transport stream function (Appendix B) was applied to a number of model disposal areas with a rectangular shape and a constant depth to obtain a qualitative evaluation of the effect of induced flow patterns on the performance and settling efficiency of these areas under steady flow and uniform or no wind conditions. The factors incorporated in this study are (a) relative location of inflow pipe and outflow weir, (b) length and number of outflow weirs, and (c) length, location, and number of spur dikes. The results of this analysis are illustrated by streamline patterns as shown in Figures 12 through 18. For the cases studied, all figures are shown approximately to scale. In each case, the quantity of water flowing between two adjacent streamlines is equal to one-tenth of the total throughput volume or discharge.

41. Confined disposal frequently takes place with both the inflow pipe and the outflow weir on the same side of the disposal area. Such a configuration and the associated flow pattern are shown in Figure 12. It can be observed that the density of streamlines is higher near the side of the area where the inlet and outlet are located. At the opposite side, and especially near the corners, waters are nearly stagnant; consequently, the effective surface area of the basin is substantially reduced with respect to that of an ideal basin where plug flow occurs.

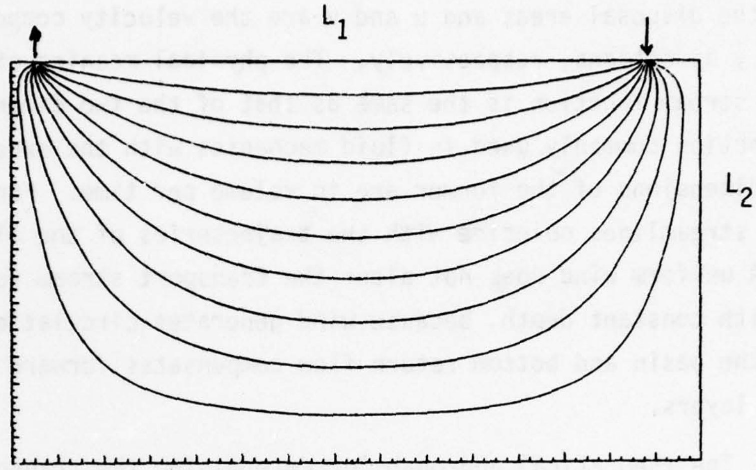


Figure 12. Streamlines for one inlet and one outlet on same side

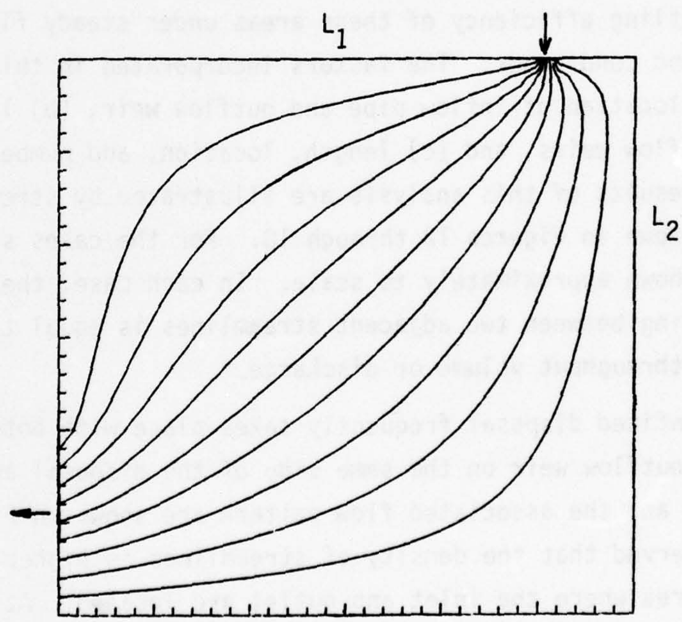


Figure 13. Streamlines for one inlet and one outlet on adjoining sides

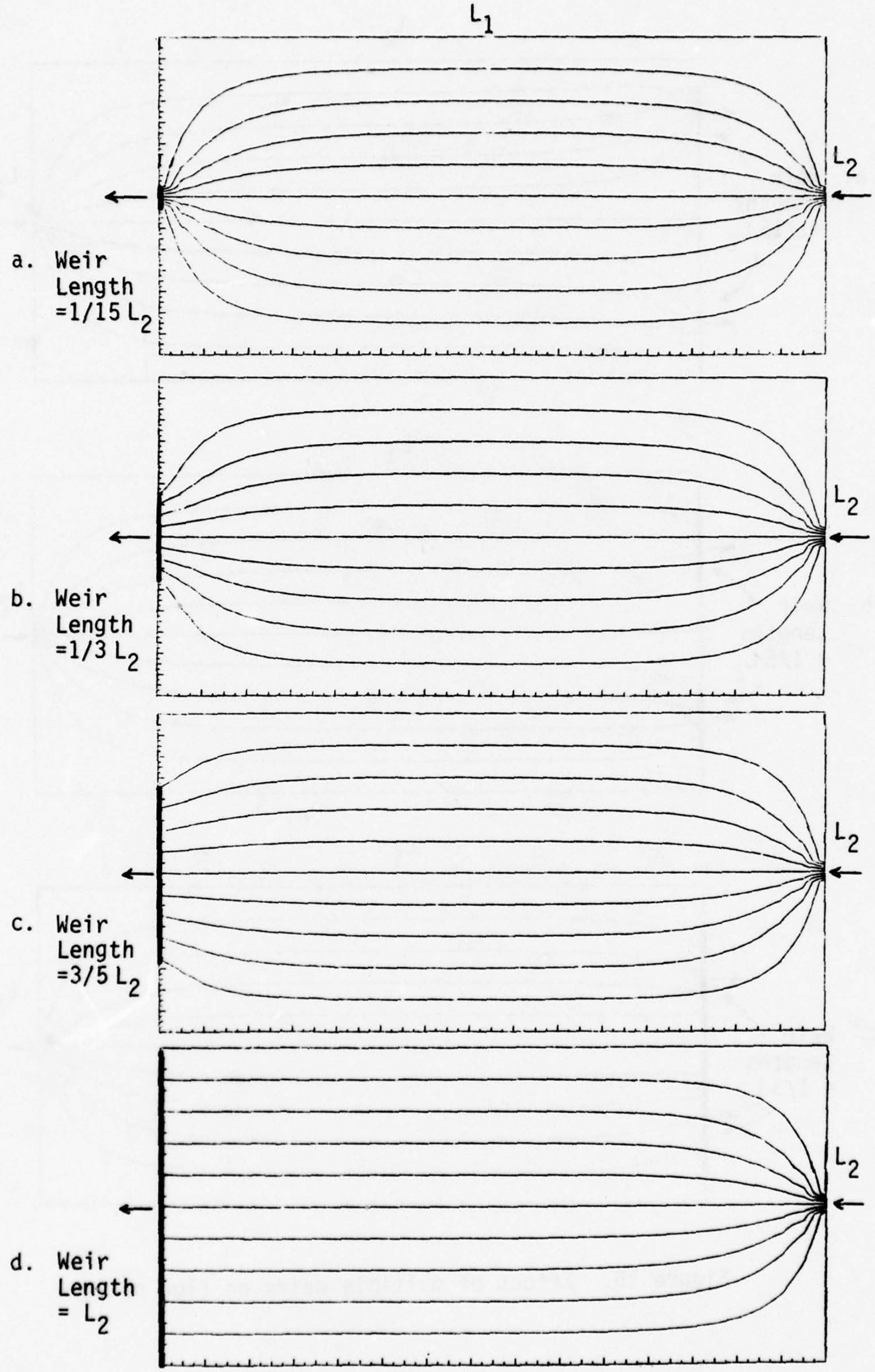


Figure 14. Effect of weir length on flow pattern

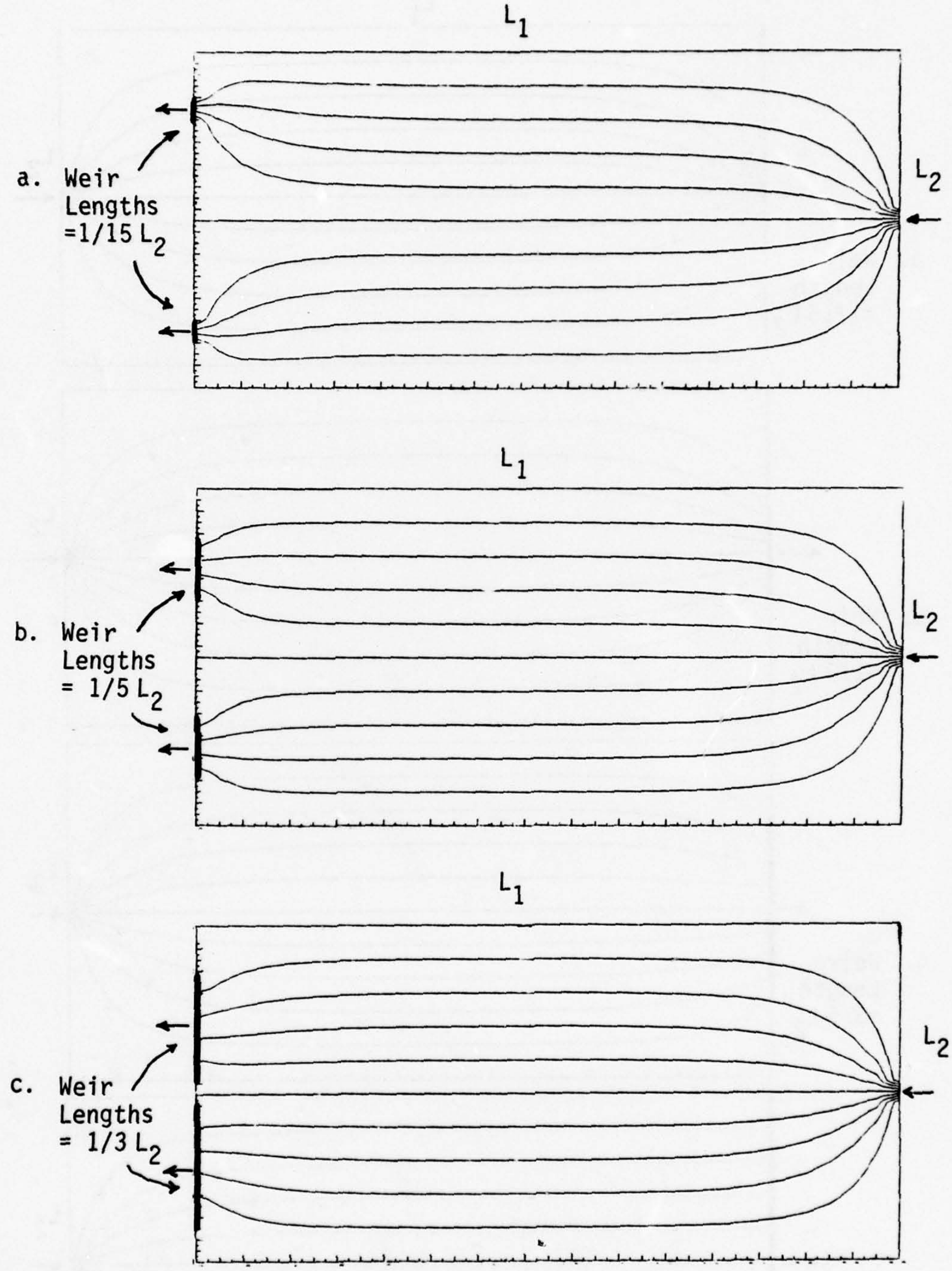
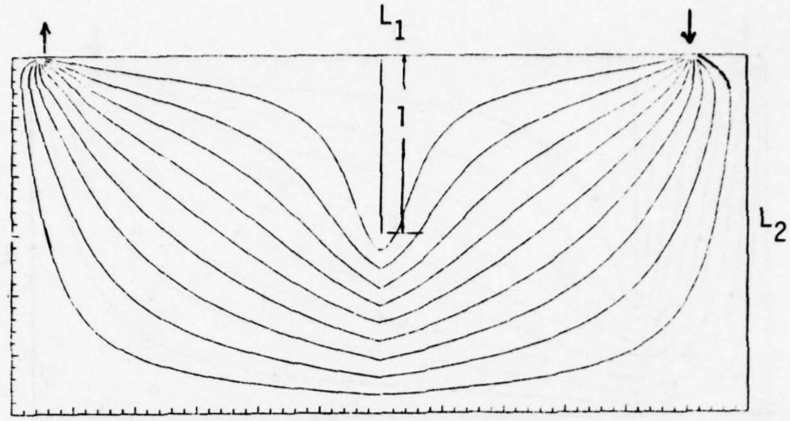
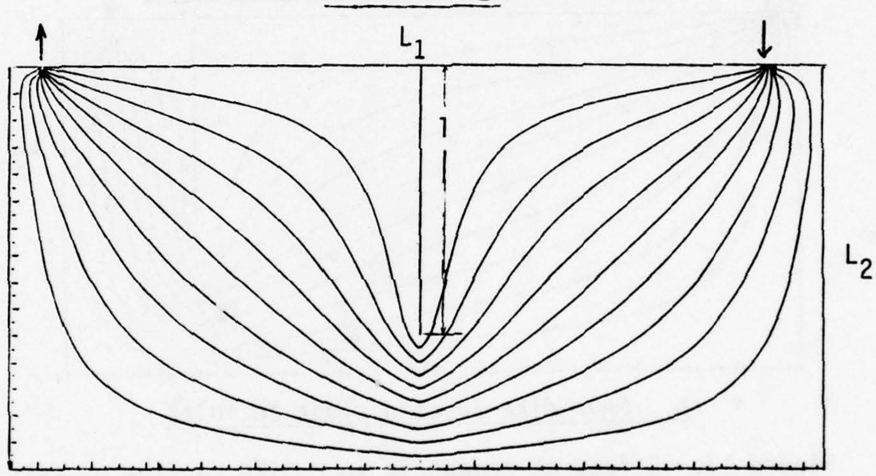


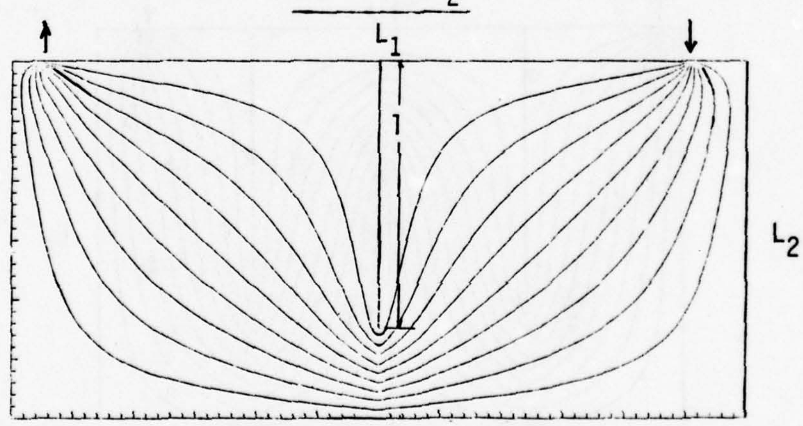
Figure 15. Effect of multiple weirs on flow pattern



a. $l = 1/2\, L_2$

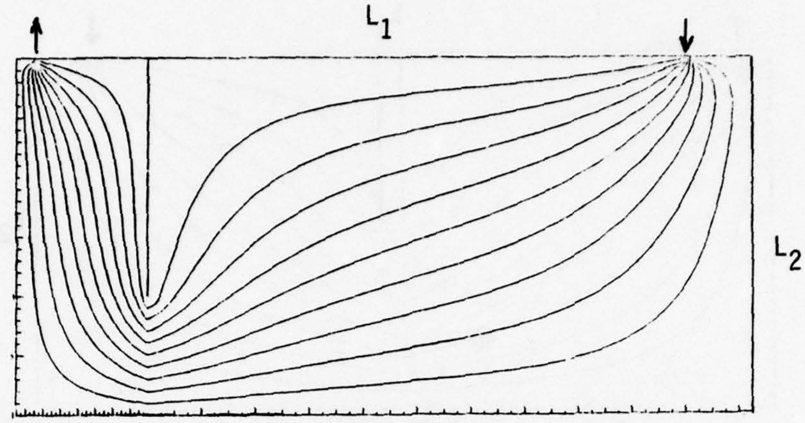


b. $l = 2/3\, L_2$

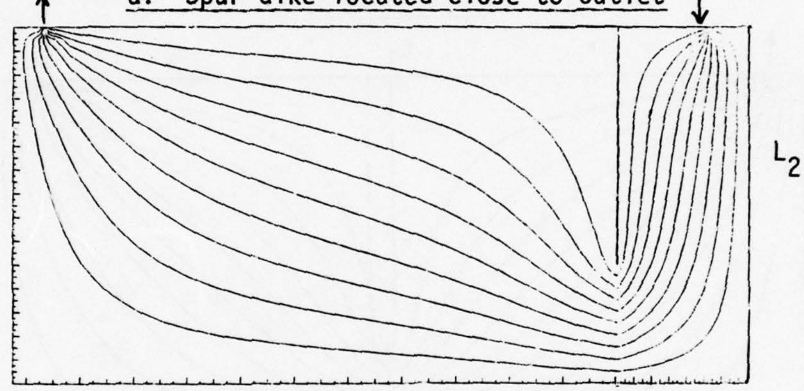


c. $l = 3/4\, L_2$

Figure 16. Effect of median spur dike on flow pattern



a. Spur dike located close to outlet



b. Spur dike located close to inlet

Figure 17. Effect of eccentric spur dike on flow pattern

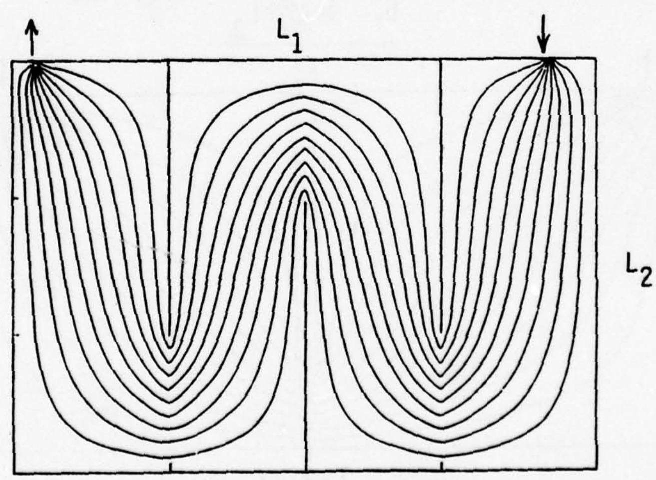


Figure 18. Effect of multiple spur dikes on flow pattern

When the inlet and outlet are located diagonally on adjoining sides of the basin (Figure 13), the effective surface area is again reduced. Although short-circuiting (higher streamline density) does occur along the diagonal of the site, it is usually not as severe as for the previous case.

42. Location of the inlet and outlet on the opposite, shorter sides of the disposal area (Figure 14) unavoidably results in some short-circuiting of the flow and reduction of the effective surface area of the basin. Variation of the weir length also has some effect on the flow pattern, but this is pronounced only in the vicinity of the weir (Figure 14). The disadvantages of short weir lengths are that (a) inactive surface area develops at the corners of the basin on the side of the weir and (b) flow velocities in the vicinity of the weir are high and this may give rise to resuspension of bottom sediments. This effect would be significant for areas with a high width-to-length ratio. The effect of multiple weirs on the flow pattern (Figure 15) is slightly better than the effect of a single weir of the same total length, and could be advantageous for areas with high width-to-length ratios. Multiple weirs would be preferred if a single weir of long length was not practical.

43. Consideration has recently been given to the use of spur dikes to improve hydraulic conditions in a disposal area. The basin shown in Figure 12 was modified to include one spur dike halfway between the inlet and outlet areas; flow patterns were determined for spur dike lengths equal to 0.50, 0.67, and 0.75 of the parallel side of the basin, and the results are shown in Figures 16a, b, and c, respectively. Short-circuiting is reduced for all of these configurations, but the effect is greater for longer spur dikes. However, spur dikes longer than 0.75 times the parallel side of the basin should not be used to avoid excessive flow concentration and increased velocities through the spur dike openings. As shown in Figure 17, a spur dike located close to the outlet weir will have a detrimental effect on the hydraulic efficiency of the basin, because higher flow velocities will occur and there will be possible resuspension of bottom sediment in the vicinity of the weir.

Spur dikes close to the inflow pipe will result in high flow velocities and suspended solids will be carried farther away, thus reducing the mounding of material and extensive sedimentation that usually take place near the discharge point. Multiple spur dikes (Figure 18) can serve to increase the retention time and minimize short-circuiting. Therefore, proper compartmentalization of a disposal area can improve its suspended solids removal effectiveness.

Wind Effects

44. The mathematical analysis of wind effects is discussed in detail in Appendix B. In order to evaluate the influence of wind on the hydraulic efficiency of a basin, consider the numerical example in which the wind velocity is 15 mph (7 m/sec) and the basin ponding depth is equal to 2, 5, or 10 feet. According to Equation B37, τ/ρ_a (wind stress/mass density of air) is approximately equal to 0.2 cm^2/sec for the 2-foot-deep basin; the surface drift velocity, $|V|$, according to Equation B37, would be about 1.8 ft/sec. From Figure B8, we obtain $\varepsilon_5/\varepsilon_2 = 2.75$ and $\varepsilon_{10}/\varepsilon_2 = 8.1$, where ε_2, ε_5, and ε_{10} are the values of the eddy viscosity coefficient for 2-, 5-, and 10-foot-deep basins, respectively. Hence, the surface drift velocity, $|V|$, will be reduced to 1.63 ft/sec and 1.11 ft/sec for basin depths of 5 feet and 10 feet, respectively. These calculations clearly demonstrate that larger ponding depths will reduce the surface drift velocity and the related possibility of sediment resuspension.

45. The velocity due to through-flow is often much smaller than the wind drift velocity. For example, consider a disposal area with a width of 400 feet and a discharge rate of 17.5 cfs. Taking an average ponding depth of 5 feet, the average velocity on any vertical section is 0.00875 ft/sec, and therefore, according to Equation B31, the surface velocity is equal to 0.013 ft/sec. However, the wind-induced surface drift velocity, $|V|$, of 1.63 ft/sec is over 100 times larger than the surface velocity due to through-flow. Thus, wind appears to be a dominant factor in the flow analysis of dredged material disposal areas.

46. Wind-generated circulation in a confined basin causes upward velocities in the windward side of the basin, as shown in Figure 19. If the wind is blowing away from the outlet weir (i. e., in the direction opposite the through-flow), this upward velocity may cause resuspension of sediments and result in poor effluent quality. Spur dikes can prevent this by changing the direction of flow perpendicular to the wind and retarding wind circulation.

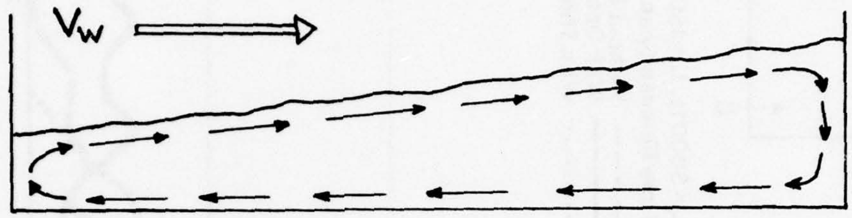


Figure 19. Sketch of wind-generated circulation in a confined basin

Retention Time

47. Using the procedures outlined in Appendix B, the distributions of retention times were determined for three rectangular basins, one without a spur dike (Figure 12), a second basin with one spur dike (Figure 16b), and a third basin with three spur dikes (Figure 18). The overall dimensions of all basins were 3000 feet by 1500 feet, the flow rate was 70 cfs, and the depth was 5 feet. A 25-hour release-time span was used, and the results are shown in Figure 20. The shortest retention time for the basin without a spur dike was 31 hours, and for the basin with one spur dike it was 41.5 hours, which is a 34 percent increase. The shortest retention time for the basin with three spur dikes was only improved another 10 percent to about 45 hours, but the dispersion curve was sharpened to more closely resemble plug flow. Retention times will not be significantly improved by additional spur dikes since the flow velocities are increased through the reduction of channel widths. Nevertheless, these results confirm quantitatively that the use of a few spur dikes decreases dispersion and increases retention time substantially. One or two spur dikes should usually be sufficient, and three or four should be the maximum number used (see Part VI).

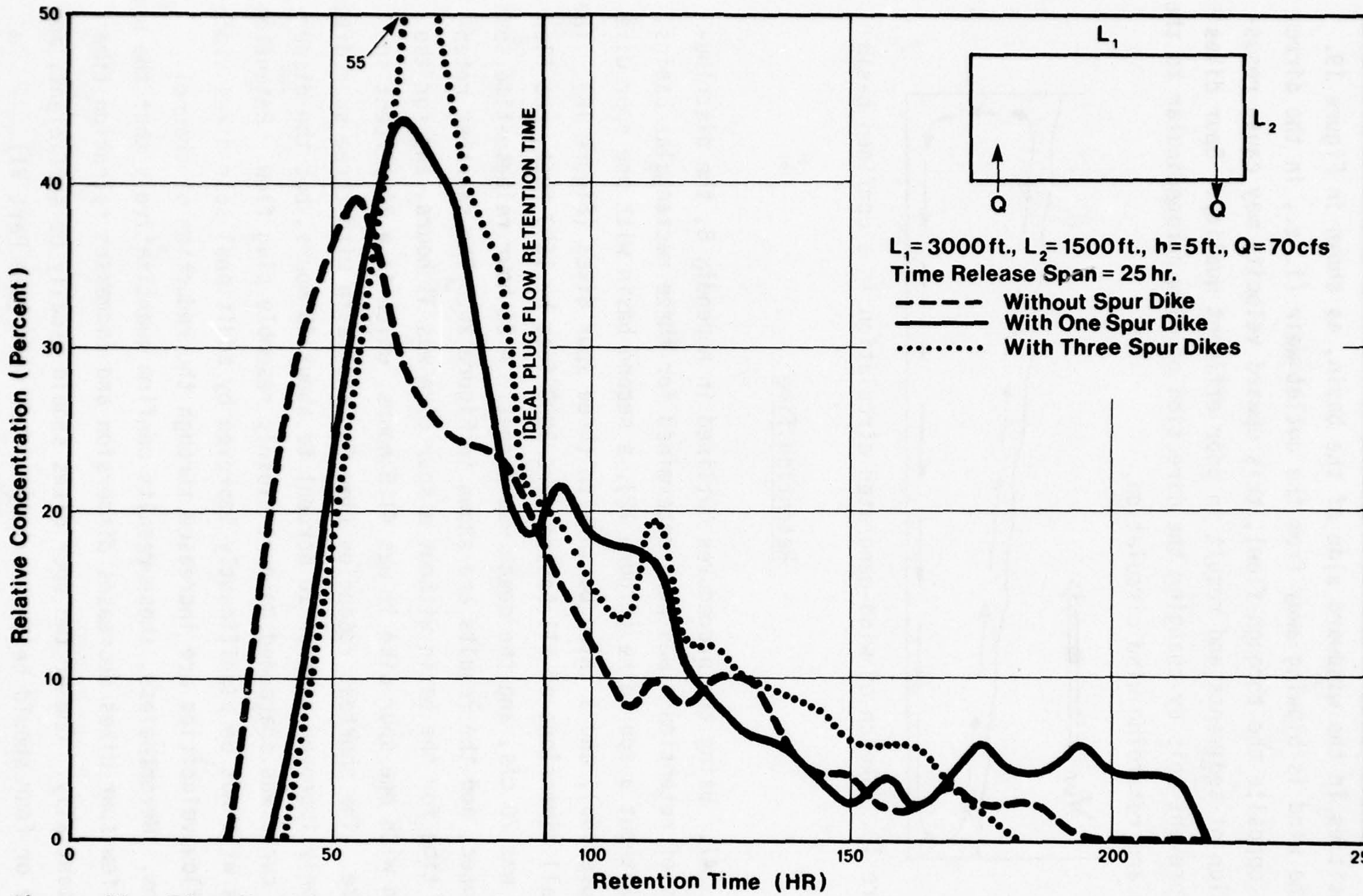


Figure 20. Distribution of predicted retention times in a model basin with and without spur dikes

48. The same procedures were applied to disposal area 5 near Yazoo City, Mississippi. This disposal area is 1700 feet long and 400 feet wide, with a discharge rate of 17.5 cfs and a ponded depth of 8 feet. The corresponding flow pattern in the basin is shown in Figure 21, and the distribution of retention times for release-time spans of 10 hours is shown in Figure 22. The model did not agree with the observed values when the actual depth of 8 feet was used. If an effective depth of 2.3 feet is used, the results fit the field data much better. This would imply that stratified flow in the basin would reduce the effective basin depth, which would reduce the basin volume and hence the theoretical retention time (in this case from 86 to 25 hours). Some of the variation between the predicted and measured curves is also due to

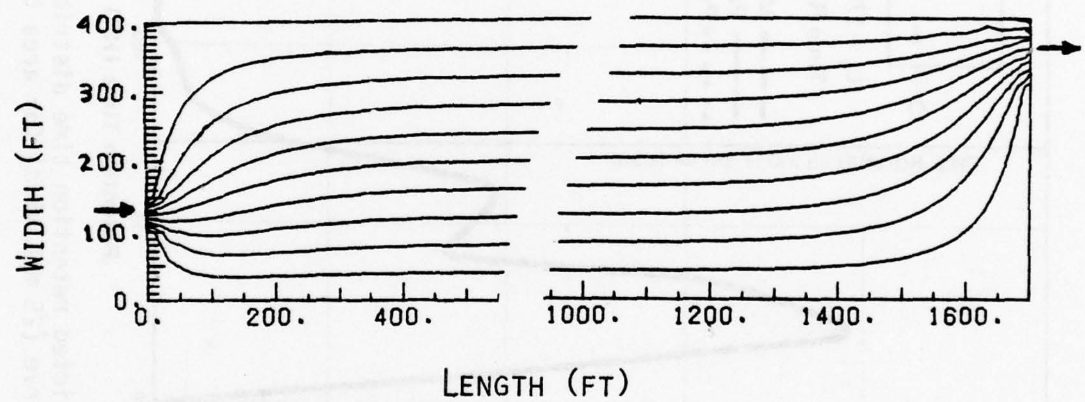


Figure 21. Predicted flow pattern for disposal area 5, Yazoo City, Mississippi

wind effects which would move the peak closer to the vertical axis and flatten the curve, and intermittent operation which would change the value used for flow and hence the retention time.

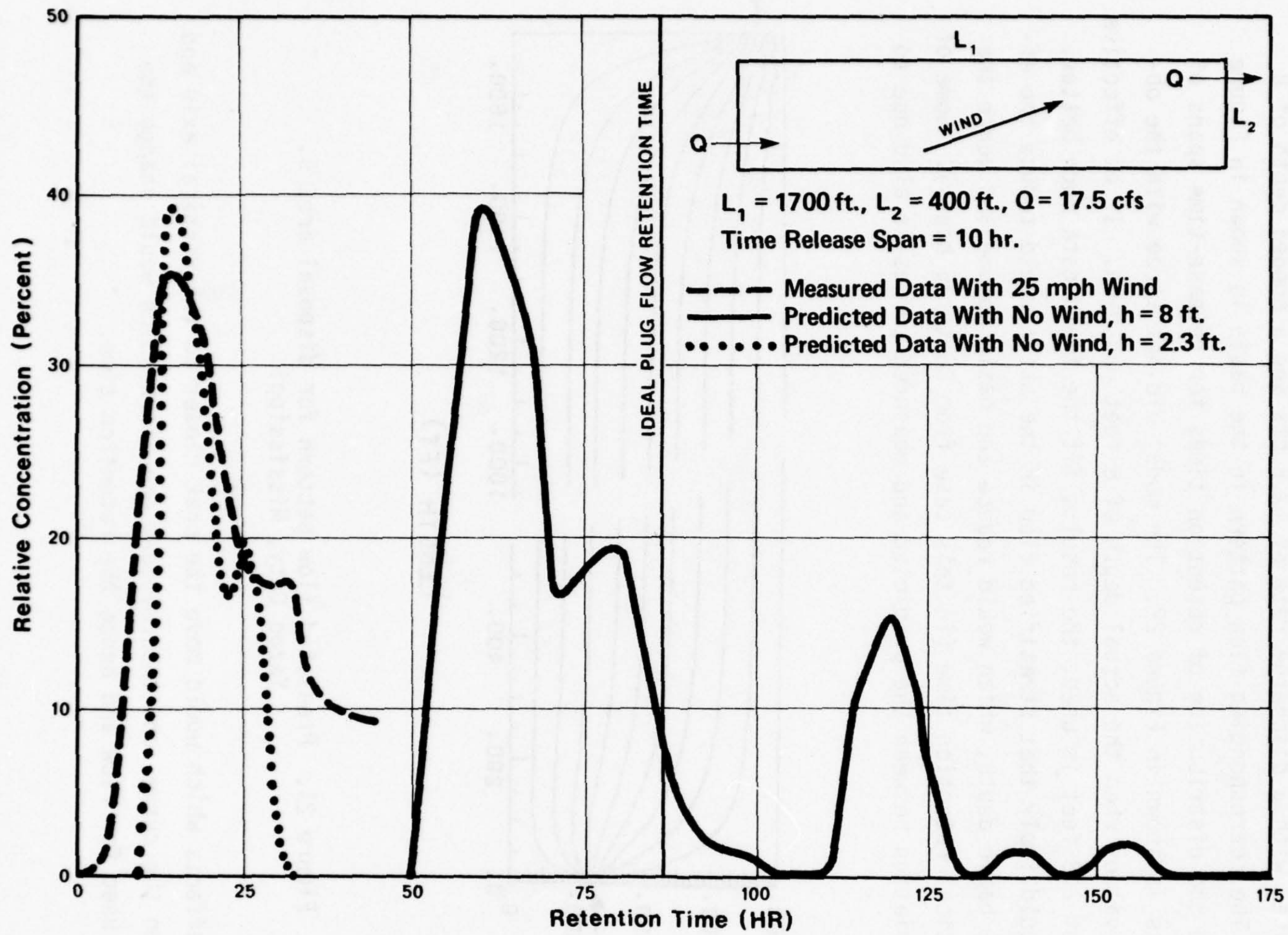


Figure 22. Predicted retention time distribution (no wind) and measured dispersion curve (25 mph wind) for area 5, Yazoo City, Mississippi

Conclusions

49. On the basis of the preceding analysis and discussion, the following conclusions can be advanced:

- a. Disposal areas should have high length-to-width ratios to maximize the effective surface area; this can be effectively obtained in square-shaped areas by the addition of spur dikes.
- b. The use of a limited number of spur dikes will increase the retention time in a basin and reduce the effects of short-circuiting; long spur dikes are more effective than short ones.
- c. Basins should be oriented with their flow patterns perpendicular to the prevailing wind. The use of spur dikes will also retard wind-induced circulation and reduce resuspension effects.
- d. Increased ponding depths provide longer retention times and reduce adverse wind effects by decreasing wind-induced circulation and surface drift velocities.
- e. Weir length does not appear to affect the retention time of a basin; however, large weir lengths are desirable to avoid high approach velocities and possible withdrawal of layers containing high concentrations of solids.
- f. If a large single weir is not practical, then multiple weirs on one side of a disposal area of equivalent total crest length can be used advantageously with essentially the same effect on the flow pattern.

PART V: DISCHARGE OF SUPERNATANTS

50. The vast majority of dredged material containment areas are equipped with some kind of sluicing device through which supernatants are discharged. The design of containment area overflow weirs is highly empirical, and pertinent guidelines are practically nonexistent. However, weir design and operation can substantially influence the hydraulic efficiency of containment areas; therefore, these effects must be carefully considered. Weirs directly affect the flow patterns of containment areas, both horizontally and vertically; but this effect is most pronounced near the weir. The manner in which supernatants are released from a sedimentation basin affects the velocity and density distributions of the water column in front of the weir, which then influence the quality of the discharged supernatants. This part of the report discusses these effects and other considerations of weir design as applied to containment areas. A general and more detailed review of weirs and other factors affecting the flow characteristics of sedimentation basins is presented in Appendix C.

51. Selective withdrawal is a concept of controlling the quality of waters released from an impoundment (see Appendix C) which may have importance in the design of weirs used for containment areas. During this study, the selective withdrawal concept was initially investigated for possible application to containment area weir design. Only preliminary findings and recommendations will be discussed in this report, since refinement of this concept and detailed design procedures will be in a "Weir Design to Maintain Effluent Quality from Dredged Material Containment Areas" report (Walski and Schroeder, in press).

Weir Parameters and Hydraulic Efficiency

52. In general, the ideal settling conditions in a sedimentation basin are hampered by a number of factors which include (a) the physical and chemical characteristics of the suspension, (b) the occurrence of

short-circuiting, (c) the resuspension of sediment, (d) the nonuniform deposition of sediment, and (e) the occurrence of turbulence in the basin. With the exception of the first factor, all others are influenced to a variable extent by the functioning of the outlet structure (overflow weir).

Short-Circuiting

53. The effect of short-circuiting becomes increasingly dominant as the inlet velocities increase; under such conditions concentration of flow develops and part of the influent passes through the basin without remaining for a sufficient period of time. Consequently, waters are discharged with inadequately settled solids. As discussed in Appendix C, experiments performed in settling tanks of various shapes indicate a close relationship between tank shape and hydraulic efficiency. If the outlet weir is contracted, the flow approaching the weir will concentrate and, depending on the degree of contraction (type and physical dimensions of weir), dead zones of considerable extent will develop; this situation will, in turn, increase short-circuiting and decrease hydraulic efficiency. Improper location of the outlet weir with respect to the inlet structure has an even more significant impact on short-circuiting and hydraulic efficiency, and this particular design consideration was discussed in Part IV.

Resuspension of Sediment

54. Resuspension of sediment is a major factor which decreases the quality of discharged supernatants. Presently available approaches to determine the conditions favorable for resuspension of sediment are not reliable enough to allow the development of quantitative criteria. However, qualitative analyses indicate that areas of flow concentration will result in bottom scouring (i. e., resuspension of sediment). Since the outlet structures used in dredged material disposal sites are usually contracted weirs, flow concentration of varying degree should be

expected. Unless flow concentration is held to a practical minimum resuspension of sediment by flow approaching the weir will occur, with an undesirable deterioration in the effluent quality. If extensive flow concentration develops (this is strongly influenced by the physical size and location of the outlet weir), sediment resuspension may have a significant impact on effluent quality.

Nonuniform
Deposition of Sediment

55. The length and location of a weir influence indirectly the uniformity of sediment deposition. Insufficient weir length and/or improper weir location give rise to short-circuiting and flow channelization and, consequently, to undesirable nonuniform sediment deposition.

Turbulence

56. The relationship between the nature of the outlet structure and the presence of turbulent effects is also indirect. As the approach velocity toward the weir increases due to contraction or disadvantageous weir location, the amount of fine particles which remain in suspension increases with a corresponding deterioration in the effluent quality.

Weir Design

57. Type, physical dimensions, and location are the major factors considered in weir design. Presented in Appendix C are various empirical, semi-empirical, and theoretical methods for the design of rectangular, polygonal, and shaft-type weirs; and discussions are offered on the factors which affect their performance. In the following paragraphs, the abovementioned weir types are addressed separately and suggestions are advanced with respect to the design of such weirs in order to improve the hydraulic efficiency of disposal areas.

Rectangular Weirs

58. Rectangular weirs are the most common outlet structure and are characterized as (a) sharp-crested or broad-crested, depending on

the thickness of their cross section; (b) with or without side contractions, depending on the ratio of weir length to channel width; and (c) free or suppressed flow, depending on the level of the downstream water body. Sharp-crested weirs have the cross section of a thin plate; weirs without contraction have lengths equal to the channel width; and for free flow weirs, the downstream water level is lower than the weir crest elevation and does not affect flow rates.

59. To simplify the hydraulic design of rectangular sharp-crested weirs without contraction, Figure 23 was developed by use of the methods documented in Appendix C. It can be observed that the head, H, affects substantially the rate of flow over the weir. When the height of the weir, P, or the ponding depth in the vicinity of the weir is assigned values larger than 1 foot, its effect on the rate of flow over the weir is negligible. With the exception of overflow structures with very long crests, weirs in disposal areas should be considered contracted; and unit flow rates determined from Figure 23 should be corrected for the effect of contraction by multiplying with a contraction coefficient which is smaller than unity. This coefficient is a function of the ratio of the head over the weir, H, to the height of weir, P, and the ratio of the weir crest length, L, to the width of the channel, B. When realistic values are assigned to the ratio H/P (not more than 0.5 and usually less than 0.1), the required correction is negligible for all practical purposes. Thus, rectangular, sharp-crested weirs in disposal areas can be designed with confidence by use of the relationship depicted in Figure 23.

60. Most rectangular weirs used in sedimentation ponds can be considered as sharp-crested weirs, despite the fact that they are, in effect, narrow-crested weirs. This assumption causes some error in the calculations, but it is insignificant, because outlet weirs are used as flow-control rather than flow-measuring devices. For this reason, Figure 23 can be used also for narrow-crested weirs. The majority of dredged material confinement areas are expected to have either sharp-

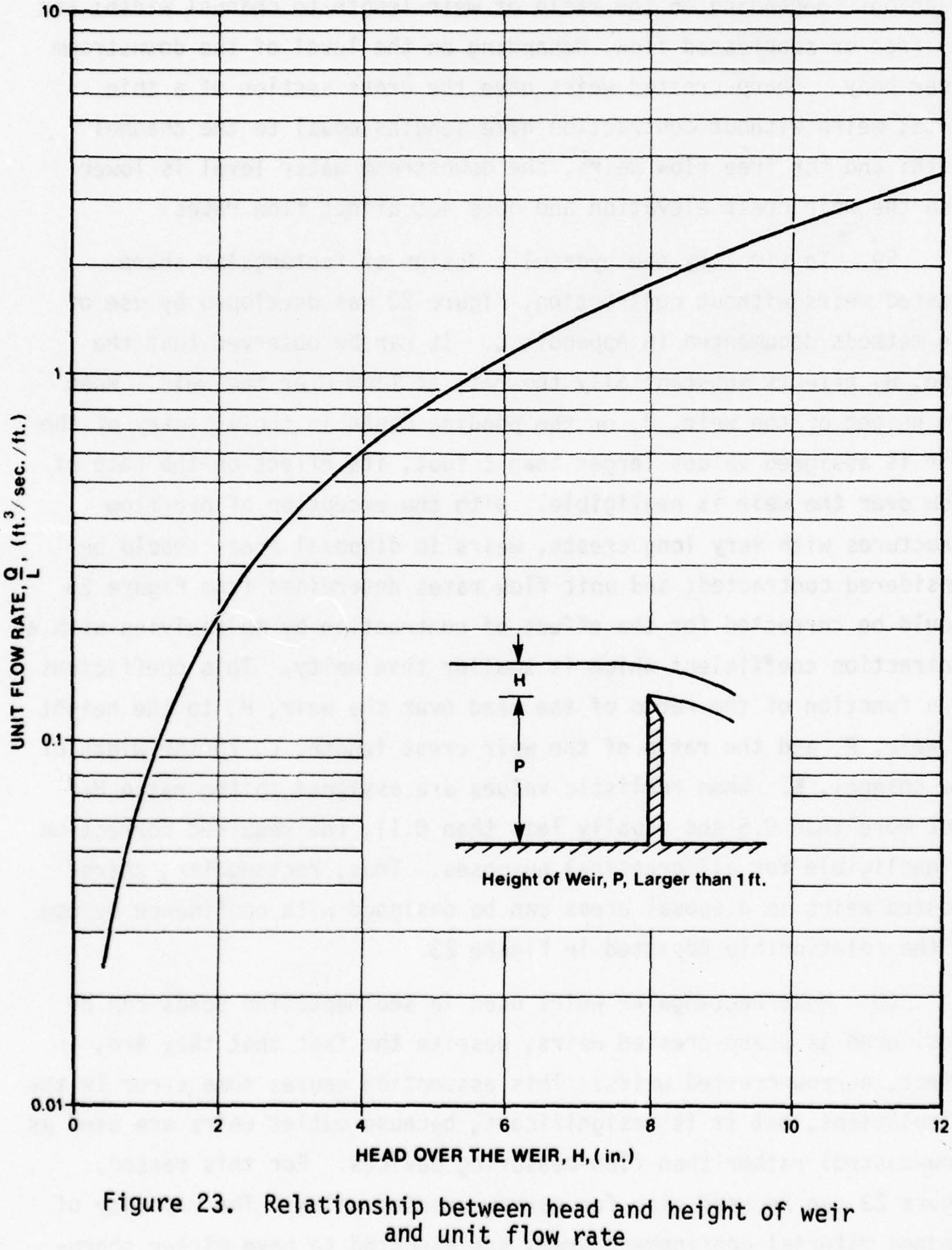


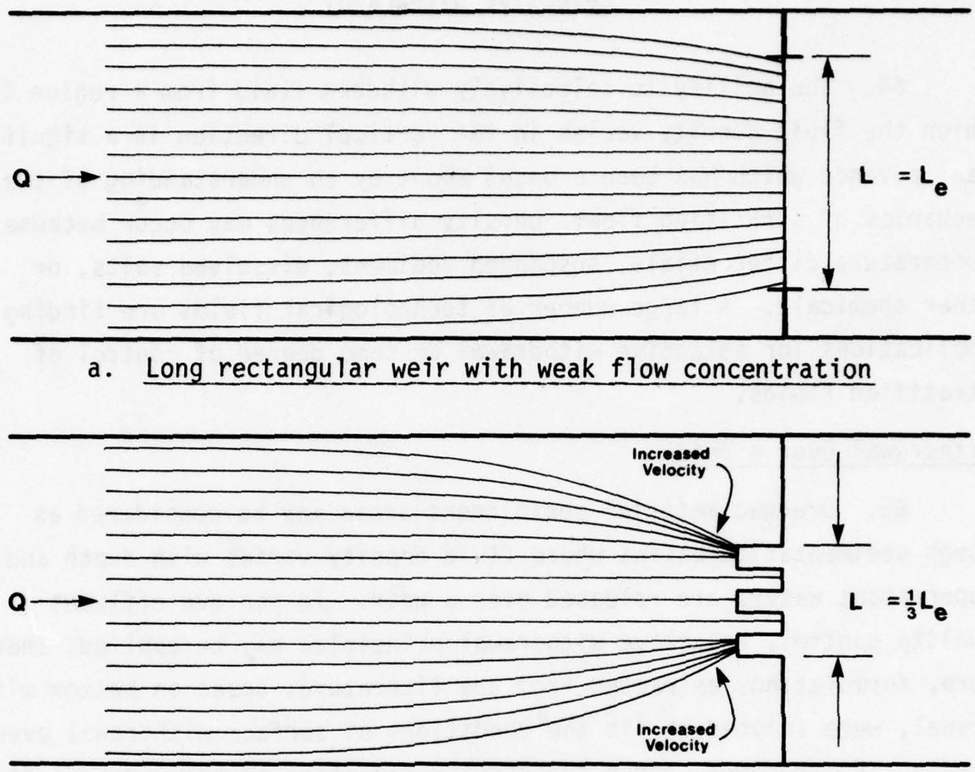
Figure 23. Relationship between head and height of weir and unit flow rate

crested or narrow-crested weirs, and this figure can be directly applied for their design. For cases where broad-crested weirs are constructed, Equations C8 and C9 can be applied.

61. To completely design a rectangular weir, the location, length, head, and height of the weir should be selected. As far as weir location is concerned, it is desirable that the outflow and inflow structures be located in such a way that the flow path is as long as possible within the existing physical limits. Weir length is an important factor in controlling short-circuiting, sediment resuspension, and overall flow pattern. Longer weir crests result in less contraction of the flow pattern (lower and more uniform flow velocities) and contribute substantially to the hydraulic efficiency of the basin (Janiak, 1976). U. S. Army Corps of Engineers Districts have recognized the significance of this factor, but have been using the head over the weir as a controlling parameter. If, for instance, the head over the weir is limited to two inches, the required weir length will be equal to the ratio of the total flow rate divided by the weir loading (flow rate per unit length), as obtained from Figure 23. The relative simplicity of this approach explains its popularity, but it has serious limitations. The weir loading itself does not have a direct effect on the hydraulic efficiency. Furthermore, the height of the weir (ponding depth) is also a significant factor that influences the flow rate. If this height is very small, the approach velocity to the weir will be high, and resuspension of sediment near the weir will occur. Thus, a two-inch head limit may be insufficient if the height of the weir is too small, while a higher head can be accepted (with a corresponding decrease in weir length) if the height of the weir is sufficient. No relationship between these factors has been established so far. In this project an attempt has been made to apply the principle of selective withdrawal to establish such a relationship, which can be used to determine the necessary ponding depth in terms of weir loading and sediment characteristics, and the results are presented in subsequent paragraphs.

Polygonal Weirs

62. Weirs of polygonal plan geometry are used frequently as outlet structures in containment areas. The purpose of a polygonal shape is to increase the active crest length, as explained in Appendix C (Figure C7). A polygonal arrangement makes it possible to increase the discharge per unit length (loading) of the weir for a given head or to decrease the depth of flow over the weir for a given total discharge. While this concept may be useful for large spillways, it is doubtful that it is appropriate for dredged material disposal areas. A decrease in the weir loading by utilizing polygonal shapes cannot and does not improve hydraulic efficiency. Figure 24 can be used to illustrate the problem. Sketch (a) shows a rectangular weir of effective length equal to the actual width of the weir; sketch (b) shows a polygonal weir of the same effective length as the rectangular weir, but contracted to a width equal to 1/3 of the actual width of the rectangular weir. In other words, although both weirs have the same <u>effective</u> length and same weir loading, the polygonal weir has been <u>compressed</u> longitudinally into a size one-third as long as the rectangular weir. Assuming that the head in both cases is small relative to the height of the weir, both outlet structures will carry the same total discharge at the same head over the weir. The cross section of the flow toward the weir is significantly smaller for the polygonal structure than for the wider rectangular structure and, therefore, the approach velocity for the former will be three times that of the latter. The higher velocity associated with the polygonal structure would then create favorable conditions for the resuspension of sediment or for the development of short-circuiting. Hence, polygonal weirs (as compared to rectangular weirs of equal total crest length) have a detrimental effect on the hydraulic efficiency of sedimentation basins. This implies that a limitation on the head over a weir may not be an effective criterion to guarantee maximum hydraulic efficiency and acceptable effluent quality. It is clear that, under identical hydraulic conditions and sediment load, a long rectangular weir will result in a significantly higher efficiency than a polygonal weir of the same effective length.



a. Long rectangular weir with weak flow concentration

b. Polygonal weir with strong flow concentration

Figure 24. Effect of polygonal weir on flow and approach velocity

Shaft-Type Weirs

63. Shaft-type weirs, such as box weirs and riser pipes, are used frequently in dredged material containment operations. Standard box weirs do not appear to be very effective in improving hydraulic efficiency for reasons identical to those advanced for polygonal weirs. Box weirs function as point sinks and force flow lines to concentrate in the area of the weir; the approach velocities are considerably increased and favorable conditions for short-circuiting and sediment resuspension are developed. Hence, compared to rectangular weirs, shaft-type weirs appear to be inferior as far as hydraulic efficiency is concerned.

Selective Withdrawal

64. The ability to selectively withdraw fluid from a region in which the fluid density varies in the vertical direction is a significant advance which has been brought about by an understanding of the mechanics of stratified flow. Density differences may occur because of temperature differentials, suspended sediment, dissolved salts, or other chemicals. A large number of technological fields are finding applications for selective withdrawal or some degree of control of stratified fluids.

Withdrawal Over a Weir

65. Dredged material containment areas may be considered as rough sedimentation basins where fluid density varies with depth and supernatant waters are released over a weir. To achieve effluent quality control, selective withdrawal principles may be applied; therefore, formulations extracted from the literature, based on bottom withdrawal, were adapted to fit the conditions of surface withdrawal over a weir. Furthermore, since the density variation in the vertical direction is unknown, a simplified assumption was made. The fluid is assumed to consist of two layers, the upper layer being acceptable for discharge while the lower layer is not. The boundary between these layers can be defined as the level where the fluid density increases (due to suspended solids, primarily) to levels higher than those dictated by the acceptable effluent quality standards; this boundary can be referred to as the interface. The withdrawal zone can be considered to extend downward to the level below which the flow velocities are minimal, if not zero, so that scouring or resuspension of bottom deposits due to turbulent eddies does not occur. Ideally, the withdrawal zone will not extend below the interface. Shown schematically in Figure 25 are the withdrawal zone and flow characteristics in the vicinity of a free-flow, sharp-crested, rectangular weir in a disposal area.

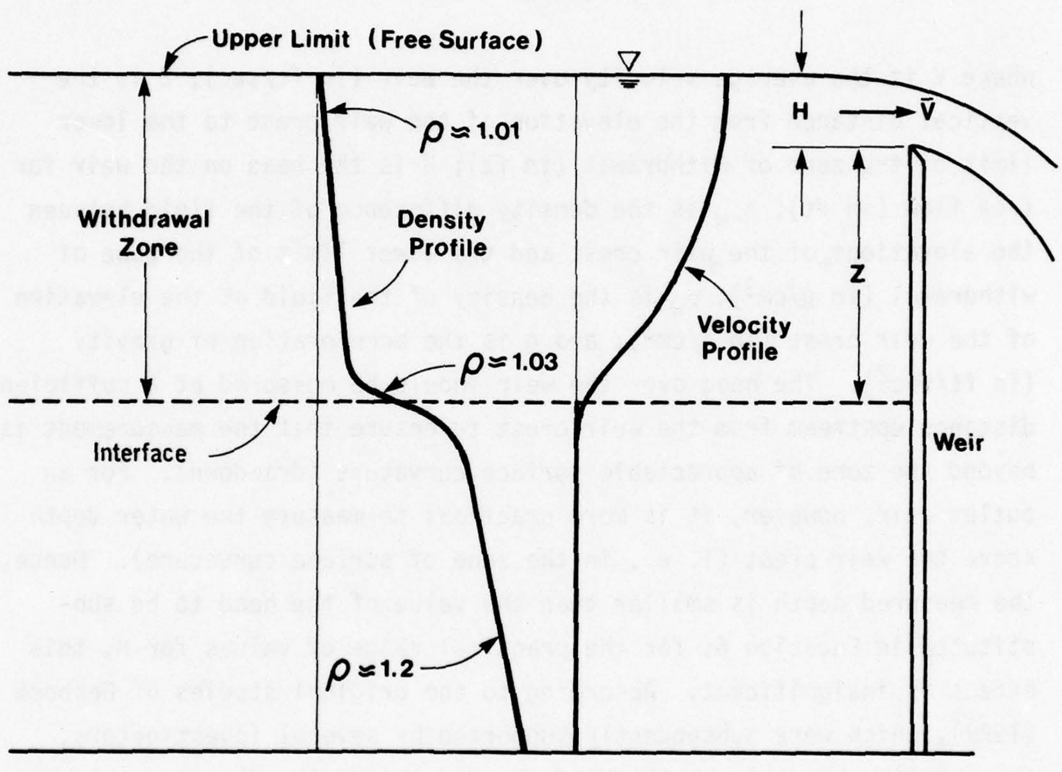


Figure 25. Withdrawal zone and fluid density profile with typical values as an example

66. An extensive literature review (Appendix C) indicates that the withdrawal zone characteristics under the abovementioned conditions and assumptions can best be described by the following relationship (Bohan and Grace, 1973):

$$\bar{V} = 0.32 \left(\frac{Z + H}{H}\right) \sqrt{\frac{\Delta\rho_w}{\rho_w} g Z} \qquad (6)$$

where $\bar{V}$ is the average velocity over the weir (in ft/sec); Z is the vertical distance from the elevation of the weir crest to the lower limit of the zone of withdrawal (in ft); H is the head on the weir for free flow (in ft); $\Delta\rho_w$ is the density difference of the fluid between the elevations of the weir crest and the lower limit of the zone of withdrawal (in g/cm^3); ρ_w is the density of the fluid at the elevation of the weir crest (in g/cm^3); and g is the acceleration of gravity (in ft/sec^2). The head over the weir should be measured at a sufficient distance upstream from the weir crest to ensure that the measurement is beyond the zone of appreciable surface curvature (drawdown). For an outlet weir, however, it is more practical to measure the water depth above the weir crest (i. e., in the zone of surface curvature). Hence, the measured depth is smaller than the value of the head to be substituted in Equation 6; for the practical range of values for H, this effect is insignificant. According to the original studies of Rehbock (1929), which were subsequently supported by several investigators, the relationship between the head, H, and the depth of water measured above the weir crest, h, is

$$H = 1.18 h \qquad (7)$$

and the measured value of h has to be adjusted to obtain H.

67. Figure 26 was prepared by use of Equation 6 to illustrate the important relationships involved in the selective withdrawal

concept. Note that, for convenience, the measured depth, h, above the weir was used rather than true head, H, and the velocity scale has been modified accordingly. To perform the necessary computations, wide ranges were selected for the parameters involved in Equation 6. Furthermore, the acceleration of gravity was set equal to 32.2 ft/sec^2, and the density of the fluid at the elevation of the weir crest was set equal to 1.00 g/cm^3 (realistic deviations from these values would have a minimal effect on the results). From an inspection of Figure 26 and Equation 6, it can be seen that the thickness of the withdrawal zone increases with increasing head (velocity) over the weir, and decreasing density difference. The velocity and head over the weir are interrelated parameters, and one does not change without a change in the other. In developing Figure 26, subcritical flow conditions (see Appendix C) were assumed upstream from the weir.

68. To obtain an understanding of the relations between the parameters shown in Figure 26, consider the case where the density difference decreases by an order of magnitude; the head over the weir should decrease by a factor of about two in order to retain the same withdrawal zone thickness. However, in many field operations it may be difficult or it may require substantial time before adjustments in the weir crest elevation can be made to accommodate new density difference conditions. Consequently, for the case where the velocity and head over the weir remain constant, the depth of the withdrawal zone would increase by a factor of about two, and this could result in a change in the effluent quality. It becomes apparent from the above simple presentation that the density profile and its variations should be well known for any application of Equation 6 to be successful.

Application to Dredged Material Disposal Areas

69. To apply the principle of selective withdrawal to dredged material disposal areas, the density profile of the waters in the vicinity of the weir should be known. However, specific density

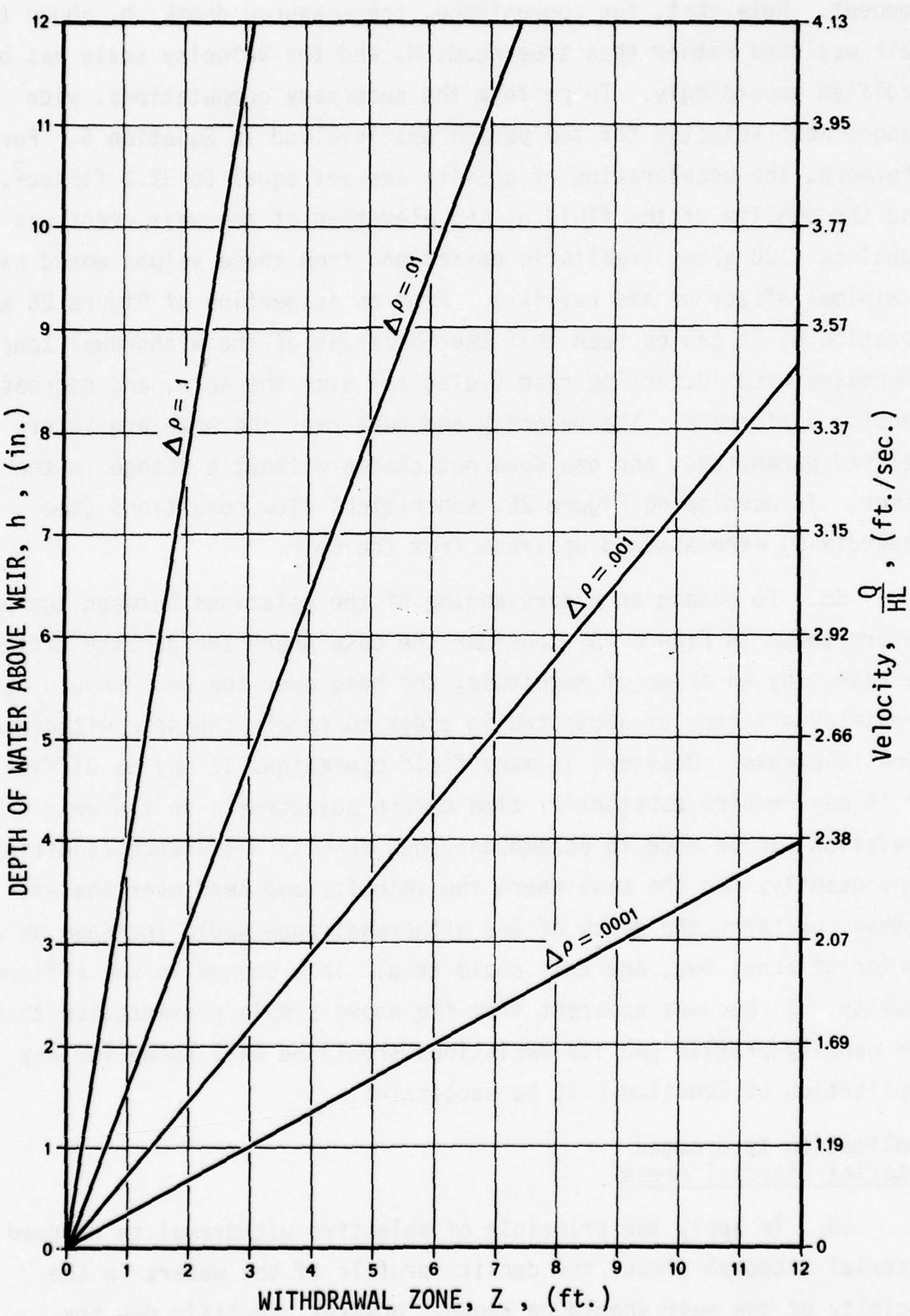


Figure 26. Relationships between depth of withdrawal zone and head over weir for various density differences

information is usually not available, and values can only be approximated from available data and experience. The density of predominantly fine-grained dredged material in disposal areas is reported to range from about 1.40 g/cm^3 to 1.65 g/cm^3 (Krizek and Salem, 1974; Lacasse, 1977). Krizek, Roderick, and Jin (1974) reported densities of about 1.20 g/cm^3 for freshly deposited dredged material during laboratory quiescent settling tests; these tests were conducted on samples of dredged bottom sediments which were not fractionated to separate the clay and silt portion from the coarser material. However, waters approaching a weir carry only the finer portion of the dredged material slurry, and this material would be expected to have an even lower density when freshly deposited. Over such a freshly deposited layer in a sedimentation basin, there exists another layer where grains are still settling from suspension, but the density (or the concentration of suspended solids) of this layer would be higher than the average density of the overlying water. The removal efficiency of disposal areas acting as sedimentation basins ranges from very poor to excellent, but values lower than 90 percent should be seldom encountered when sites are properly designed and managed. Considering that the average concentration of suspended solids in the influent slurry is about 15 percent by weight, than the amount of suspended solids in the waters approaching a weir should not exceed 1.5 percent by weight (about 15 g/l or a density of 1.01 g/cm^3) and would often be much lower. Recent samples collected 3 to 5 feet below the water surface near the weir of an active disposal site indicated densities ranging from about 1.005 to 1.03 g/cm^3 (representing suspended solid concentrations from about 8 to 48 g/l).

70. According to the limited information presented above, an accurate density profile for a specific weir site should be developed. On the basis of this density profile, selective withdrawal principles can be applied (a) to withdraw waters with acceptable quality and (b) to avoid resuspension or scour of bottom sediments. To achieve this effect, the withdrawal zone should be located well within the upper layer of water and flow velocities should be minimal, if not zero, in

the second water layer. If accurate density profiles are not available, Figure 26 can be used qualitatively as an aid in selecting appropriate crest elevations and ponding depths to maintain proper effluent quality. For example, a density difference of about 0.01 to 0.02 g/cm^3 over the withdrawal zone is a reasonable estimate to be used for preliminary planning. $\Delta\rho$ = 0.01 is equivalent to a suspended solids concentration difference of about 16 g/l. From Figure 26, it can be seen that for $\Delta\rho$ = 0.01, the withdrawal zone increases from about 1 to 4 ft as the depth of water over the weir increases from 2 to 6 inches; and these represent operational values frequently encountered. Figure 26 can be used to select a minimum ponding depth to maintain a withdrawal zone for a specified weir loading; or conversely, to limit the weir loading for a desired ponding depth. In both cases, a margin of safety should be used in selecting operational values to ensure proper performance.

Conclusions

71. Based on the foregoing information, the following conclusions can be advanced:

- a. The operating conditions of an overflow weir can substantially affect the hydraulic efficiency of a dredged material confinement area and the quality of the discharged supernatants.
- b. Weirs should be strategically located to minimize flow contraction and maximize the length of flow lines.
- c. Long, sharp-crested rectangular weirs appear to be the most promising candidates to improve the hydraulic efficiency of disposal areas.
- d. Selective withdrawal principles should be applied to estimate the necessary ponding depth in the vicinity of a weir to avoid withdrawal of layers containing high concentrations of solids.

e. Large ponding depths in the vicinity of the weir and low water heads at the crest of the weir increase the efficiency of a disposal area and improve the quality of discharged supernatants.

f. A detailed weir design procedure can be developed to maintain specific effluent quality if density profiles are known; and such a procedure will be forthcoming as a result of related investigations.

PART VI: ECONOMIC CONSIDERATIONS

72. The economic considerations of maximizing hydraulic efficiencies of disposal areas involve many factors such as type and quantity of material to be dredged; availability of disposal areas; characteristics of area soils; and desired quality of effluents. As described in Appendix A, disposal costs represent about 15 to 20 percent of the total project costs, and will probably increase in the future. The following economic considerations are presented to help the designer minimize disposal costs while maximizing hydraulic efficiencies. However, these considerations are frequently interrelated with the overall dredging project factors such as hauling or pumping distances to disposal areas, which are not part of this analysis. The reader should refer to "Dredged Material Transport System for Inland Disposal and/or Productive Use Concepts" by Souder et al. (in preparation) for additional economic considerations of disposal operations.

73. The following section discusses an overall concept of evaluating disposal projects for minimum total costs; and the second section illustrates how outside dike shapes and sizes affect the unit cost of disposal operations. Finally, the third section discusses optimum internal configurations to maximize hydraulic efficiencies at minimum unit costs per cu yd. Additional details and generalized analytical equations for this part are presented in Appendix D.

Net Present Worth of Project Costs

74. The net present worth method of analyzing total project costs is frequently used to evaluate government sponsored projects. This method converts estimated costs over the project period into equivalent net present worth values which, when minimized, reflect the most economical alternative. In order to apply this method, project costs are grouped into capital costs and operation and maintenance (O & M) costs, and salvage values are determined for any assets remaining at the end of

the project period. The net present worth (NPW) of the project is determined by

NPW = capital costs + discounted O & M costs
- discounted salvage values

The future O & M costs and salvage values are discounted back to present values using an acceptable interest rate and standard discounting formulas. By applying these procedures, the net present worth of several alternative projects can be used for making equalized comparisons and selecting the most economical project based on minimum NPW.

75. Disposal costs can be grouped into categories of (a) planning and engineering, (b) land acquisition, (c) construction, and (d) operation and maintenance. With the exception of land values, most disposal projects should not have any appreciable salvage values at the final end of the project (no further use for disposal). This disregards consideration of potential reuse of dredged material and/or containment areas. Furthermore, land acquisition costs and salvage values would normally not have to be considered for applying NPW analysis to disposal projects. This is because land does not depreciate and land acquisition costs, either to the sponsor or to the government if purchased, would be offset by the reclaimable land values at the end of the project. An exception would be if the disposal project definitely affects the land value significantly, such as making it much more usable than originally. A reasonable assumption is that most land values will slightly appreciate over the project period which would offset the present value discounting when conservative interest rates are applied.

76. The result of the previous analysis greatly simplifies the NPW method for application to disposal projects. Capital costs include all planning, engineering, construction and contingency costs; and the only other consideration will be the discounted O & M costs which can have a significant effect on NPW. For example, consider a large dis-

posal area that is to be used intermittently over a 20-yr period. Assume that one design of minimum height dikes costs $300,000 to construct while an alternative design of higher, stronger dikes costs $500,000 to construct. The estimated O & M costs for the first design, including dike repairs and improvements, are $40,000 per year while the same costs are reduced to $20,000 per year for the higher, stronger dike design. Failure to consider the NPW of the O & M costs would lead to selecting the first design as the most economical project, when actually the second design has the minimum NPW. This example is based on an assumed 6-1/8 percent discount rate over 20 years and the NPW analyses are as follows:

$$\text{Design 1:} \quad \text{NPW} = \$300{,}000 + \$40{,}000\ (11.3544) = \$754{,}400$$

$$\text{Design 2:} \quad \text{NPW} = \$500{,}000 + \$20{,}000\ (11.3544) = \$727{,}100$$

A discussion of the standard discounting procedures used in NPW analysis is beyond the scope of this work. The important point is that containment area designs of minimum initial capital costs are not always the most economical over the long run. Improved dike designs, even though more expensive initially, could be the most economical if their O & M costs are sufficiently minimized.

Effects of Basin Size and Shape

77. Large square-shaped areas are more economical to construct than small square-shaped areas. This is obvious since the diked perimeter, P_a, and associated construction costs, increase according to the square root of the area. Therefore, a 1000-acre site would only cost 3.16 times more than a 100-acre site to construct, assuming both dike

perimeters are square-shaped and of the same height. This analysis does not consider the value of the land required, due to reasons previously given. In addition, the actual costs of clearing, grading, etc., could increase the overall costs considerably. Nevertheless, when dike construction costs are the primary factors considered, the economy of scale obtained with large disposal areas can be very beneficial. (See Appendix D for specific effects of area size and shape.)

78. It has been stated earlier that disposal areas should have high length-to-width ratios (elongated type shapes) for maximum hydraulic efficiency. This is in apparent conflict with economic considerations which would promote square-shaped areas or low length-to-width ratios. Methods for obtaining higher length-to-width ratios from square-shaped areas will be given next to resolve this dilemma. The designer must make a judgement on the size and shape required and the relative costs involved to maximize hydraulic efficiency, based on the types of scarce areas available. Long, narrow strips of land along waterways (when obtainable) are convenient for dredged material disposal and may have to be used. If not available, large upland areas may be very economical, but the longer pumping or hauling distances involved could cancel out any economic benefits obtained.

Optimum Internal Configurations

79. Spur dikes can be constructed by a variety of means at relatively low costs. The number and type of spur dikes to be constructed depends on the size and shape of available area, inlet and outlet arrangements, prevailing wind conditions, mounding considerations, and other factors. As a general rule, spur dikes should be used to (a) prevent short-circuiting between inlet and outlets, (b) increase effective length-to-width ratios of active settling area, (c) redistribute material to avoid mounding, and (d) change direction of flow patterns. The last mentioned use of spur dikes can be very important when prevailing winds induce undesirable circulation patterns in a settling basin

or when a return flow pattern is required to provide a convenient location for effluent release, as explained in Part IV.

80. As shown in Appendix C, the most effective type of spur dikes are longitudinal ones parallel to the long side of the basin. However, transverse spur dikes perpendicular to the long side of the basin can be very useful when the inlet and outlet are located along one side out of necessity. In either case, a minimum length-to-width ratio of approximately five should be provided for the flow pattern if possible (Janiak, 1976; EPA, 1976). The length of the spur dikes should be approximately 0.75 times the length of the parallel basin side as explained in Part IV. Several examples of different configurations of spur dikes are illustrated in Figure 27. Note that more transverse spur dikes (of shorter length) are required than longitudinal ones to obtain the same effective length-to-width ratio. Also, an odd number of spur dikes will result in inlet and outlets being located on the same side while an even number will result in inlet and outlets being located on opposite sides of the basin.

81. The additional cost of spur dikes and the loss of surface area and containment volume caused by their presence must be considered in their design. Accordingly, Tables 4 and 5 have been prepared based upon an extensive analysis of the performance and cost factors of one to four spur dikes added to basins of different shapes or length-to-width ratios (see Appendix D). The primary purpose of these tables is to assist the designer in selecting the type and number of spur dikes, N, for a basin area with existing length-to-width, M, which will change the flow pattern and produce a new, longer length-to-width ratio, M*. In addition to the new length-to-width ratios, M*, the tables show the relative increase in overall dike construction costs from the addition of spur dikes and the related loss of surface area and containment volume when they are put into the basin. The information provided in the tables is derived from a generalized mathematical analysis presented in Appendix D with specific parameters based on the following assumptions:

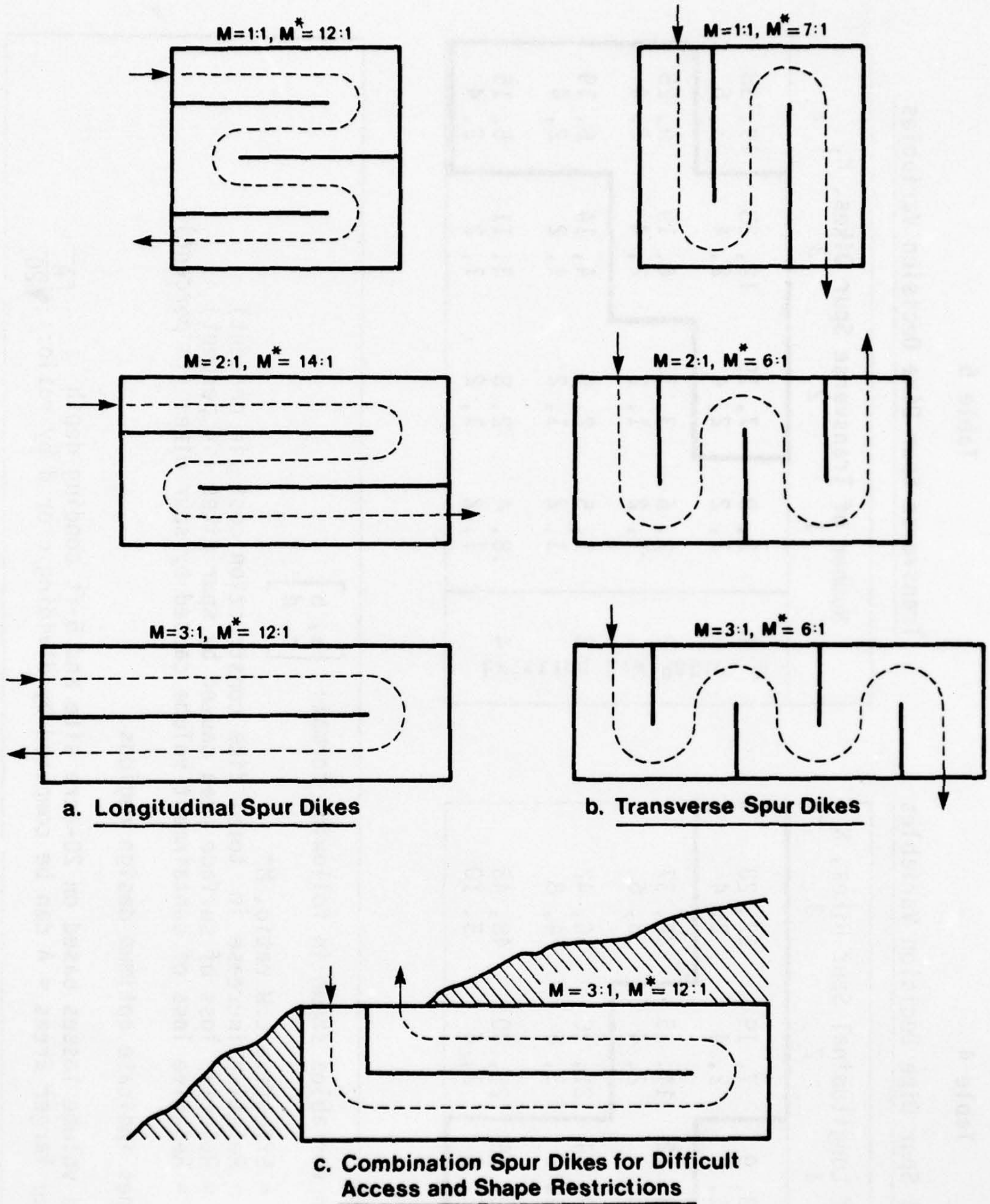


Figure 27. Examples of longitudinal and transverse spur dike configurations

Table 4

Longitudinal Spur Dike Decision Variables

Existing L:W Ratio, M	Number of Longitudinal Spur Dikes, N_L 1	2	3
1	3, 9 1, 2	7, 19 2, 4	12, 28 2, 4
2	6, 12 1, 2	14, 25 2, 4	24, 37 3, 6
3	9, 14 1, 2	21, 28 3, 6	36, 42 4, 8
4	12, 15 2, 4	28, 30 3, 6	48, 45 5, 10

Table 5

Transverse Spur Dike Decision Variables

Existing L:W Ratio, M	Number of Transverse Spur Dikes, N_T 1	2	3	4
1	3, 9 1, 2	7, 19 2, 4	12, 28 2, 4	19, 38 3, 6
2	2, 6 1, 2	3, 13 1, 2	6, 19 2, 4	9, 25 2, 4
3	1, 5 1, 2	2, 9 1, 2	4, 14 1, 2	6, 19 2, 4
4	.8, 4 1, 2	2, 8 1, 2	3, 11 1, 2	5, 15 2, 4

NOTES

1. Decision variables shown in following format: $\begin{bmatrix} a, & b \\ c, & d \end{bmatrix}$

   where a = Effective L:W ratio, M*
   b = Relative increase in total dike construction cost (in percent)
   c = Relative loss of surface area caused by spur dikes (in percent)
   d = Relative loss of containment volume caused by spur dikes (in percent)

2. Bold lines indicate optimum design regions

3. Area and volume losses based on 20-acre site and 5-ft ponding depth
   Losses for larger areas = A can be computed by dividing c or d by ratio: $\sqrt{\frac{A}{20}}$

a. The cost for constructing a spur dike (per lineal foot) will be about one-half of the cost for constructing the main perimeter retaining dikes, due to proportionally less material required.

b. The loss of area for each spur dike strip is estimated to be 10 square feet per lineal foot of dike. The proportional loss of area becomes less as the size of basin increases and the maximum area lost is less than two percent for basins greater than 100 acres.

c. The loss of volume for each spur dike is based on a spur dike volume of 100 ft^3/lineal foot and a containment depth of five feet. The proportional loss of volume also becomes less as the size of basin increases and the maximum volume lost is less than four percent for basins larger than 100 acres (the percent loss of volume is twice the percent loss of area based on the example parameters chosen).

d. The area and volume loss parameters in Tables 4 and 5 are based on a disposal area size of 20 acres to provide relative comparison data for small to moderate size disposal areas. The area and volume losses become relatively insignificant for large size basins (larger than 200 acres).

82. A careful examination of the matrix decision tables will show that the optimum benefit/cost combination of basin shape and type and number of spur dikes has been outlined by the bold lines. Combinations to the left of these lines do not provide high enough length-to-width ratios and combinations to the right become significantly more expensive and result in greater loss of usable area and volume. As explained earlier, a minimum number of spur dikes should be used since additional ones are not effective.

Conclusions

83. Based upon the analysis shown, the following conclusions are presented:

a. Containment areas of low initial capital costs are not necessarily the most economical alternative when evaluated by the NPW method if O & M costs are properly considered.

b. Large square-shaped disposal areas with two or three internal spur dikes provide the most economical approach to maximizing hydraulic efficiency.

c. Very large square areas with internal spur dikes provide significant economies of scale obtained from the overall size and shape parameters, but longer slurry transporting costs could outweigh these benefits.

d. Elongated disposal areas with length-to-width ratios of 2-4 can be most economically improved to maximize hydraulic efficiency by the addition of one longitudinal spur dike if inlet-outlet conditions permit.

e. When necessary to prevent short-circuiting due to inlet and outlets being located on the same side, transverse spur dikes can most economically be added to elongated disposal areas with length-to-width ratios M by building M + 1 transverse spur dikes up to a maximum of four.

PART VII: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

84. The hydraulic efficiency of a containment area is an important characteristic that has a direct effect on both the economy and quality of disposal site performance. The hydraulic efficiency indicates the amount of effective area being utilized in a disposal site and can be represented by the ratio of average retention time to ideal retention time. Hydraulic efficiency is affected by a multitude of factors which fall into the general categories of (a) containment area geometry, (b) ponded water depth and volume, (c) inlet and outlet structures, and (d) weather conditions. Some factors, such as the maximum height of containment dikes, vary considerably with location, and the amount of control that can be exercised on them by the designer is very limited.

Containment Area Geometry

85. Economic constraints and land use patterns generally govern the size and geometry of the land that can be acquired for use as a dredged material containment area. The required size of a containment area can be estimated when the slurry characteristics and the effluent quality standard are known, because these parameters indicate the removal efficiency to be achieved (Palermo, Montgomery, and Poindexter, in preparation). The most economical disposal sites are large, square-shaped areas, but they are hydraulically inefficient unless modified by internal configurations.

86. The shape of the containment area should be such that most, if not all, of the enclosed volume is effectively utilized for sedimentation purposes. Accordingly, it is desirable for containment areas to

have elongated rectangular shapes with flow paths parallel to the longer side of the rectangle (high length-to-width ratios). When such a shape can not be obtained, a minimum number of spur dikes can be used to increase the effective length-to-width ratio and maximize the effective surface area. However, care should be taken to guarantee that the spur dikes do not result in excessive localized contraction of the flow, because this condition would cause high flow velocities, disturb the settling processes, and increase the possibility of sediment resuspension.

87. Spur dikes appear to be the most economical method to modify available areas to provide efficient flow patterns, increase effective length-to-width ratios, minimize prevailing wind effects and/or prevent short-circuiting when inlet and outlet must be located on the same side. Spur dikes are not as expensive to build as retaining dikes since hydrostatic pressures are approximately equal on both sides and they can be constructed hydraulically with dredged material. The most economical and efficient disposal sites would be large, square-shaped areas modified with two or three internal spur dikes to obtain high length-to-width ratios of the modified flow path.

<u>Ponding Depths and Volume</u>

88. Ponding depth is an important parameter controlling disposal area performance. Basically, ponding depths should be as high as possible to provide longer retention times, minimize induced flow velocities and maximize protection against resuspension and discharge of bottom sediments. The engineering characteristics of the foundation soils and the dike material dictate the maximum safe height of the retaining dikes and, consequently, the maximum ponding depth of water in the containment area.

89. There is no real substitute for providing adequate surface areas to obtain settling effectiveness. However, if sufficient area is not available, increased ponding depths will provide greater volumes and retention times to compensate for some loss of active area, but this will usually require much better dikes to be constructed at increased costs.

Inlet and Outlet Structures

90. A pipeline of varying length is normally used to pump dredged material slurry into a containment area. To achieve better distribution and mixing of the slurry in the inlet zone of the containment area, the pipeline may be equipped with multiple outlets using Y-type connectors. When the dredged material mounds in front of the inlet pipe, it should be removed; if this is not possible the location of the inlet pipe should be changed laterally. Extending the pipeline inside the containment area should be avoided, because this aggravates the effect of short-circuiting.

91. The most suitable outlet structure for dredged material containment areas is a rectangular, sharp-crested, free-flow weir. To maximize hydraulic efficiency, weir crests should be long and the head over the weir should be small. In the vicinity of the weir, the ponding depth of the water should be high enough that resuspension of bottom sediments is avoided and withdrawal of waters of acceptable quality is facilitated.

92. The inlet and outlet structures should normally be located at the opposite, shorter sides of a rectangular area, and each structure should be constructed symmetrically with respect to the long axis of the rectangle. When the shape of an area is not rectangular or when spur dikes are used, the relative locations of the inlet and outlet structures should be chosen to maximize the length of the flow paths and the effective surface area. The addition of an odd number of spur dikes (1, 3, etc.,) allows the convenient use of one long side of the dike for both inlet and outlet devices such as frequently is required along a waterway.

Weather Conditions

93. The hydraulic efficiency of a containment area is adversely affected to a large extent by uncontrollable wind-induced currents. To avoid or minimize either resuspension or surface short-circuiting resulting from wind-induced circulation near the overflow weir, the

locations of the inlet pipe and the outlet weir should be along an axis perpendicular to the prevailing wind direction. Spur dikes can also be used advantageously to change the direction of flow and retard wind-induced circulation.

Recommendations

94. The following recommendations are presented as general guidelines for maximizing hydraulic efficiencies of disposal areas. Detailed design procedures will be forthcoming in a containment area sizing synthesis report (Palermo, Montgomery, and Poindexter, in preparation).

1. Disposal Site Selection

   a. Nearby, large, square-shaped areas in or near dredged waterways should be considered first for future disposal areas and compartmentalized with spur dikes.

   b. Long, but closeby strips of land parallel to waterways should be considered next, particularly if one or more natural constraining structures are available to minimize dike construction costs. Inlet and outlets can be conveniently located on the dike adjacent to the water if an odd number of transverse spur dikes are used.

   c. Long, narrow strips of land perpendicular to waterways can also be effectively used by the addition of one longitudinal spur dike to return the flow to the water side for discharge.

   d. If none of the above are possible, then large, square-shaped upland areas should be considered which will have to be compartmentalized. This type of site may have to be used in conjunction with rehauling areas to minimize booster pumping and other long-range disposal costs.

2. Internal Configurations

a. If the shape of the land is square or irregular and can not be approximated by an elongated rectangle, consideration should be given to the use of spur dikes; in general, the length of a spur dike should be approximately 0.75 times the parallel side of the area at the location of the dike, and spur dikes should not be located close to an overflow weir.

b. If the available tract of land has a shape that can be approximated by an elongated rectangle with a length-to-width ratio greater than about five, internal spur dikes are probably not necessary but still can be used if desired. For example, they can be used to (a) further increase the length-to-width ratio, (b) change the direction of flow to allow location of inlet and outlet on one side, or (c) change direction of flow to retard prevailing wind effects.

c. In either case, the designer should carefully evaluate different spur dike configurations by use of Tables 4 and 5 in Part VI. The minimum length-to-width ratio of the modified flow path should be five or greater.

3. Bottom Topography

a. Ridges and mounds in the containment facility reduce the size of the area for effective sedimentation and cause channelization of the flow; proper consideration must be given to this factor when estimating the effective area for sedimentation.

b. Marsh grass or other natural vegetation should normally be left in the basin as an additional filtration aid for the sedimentation process.

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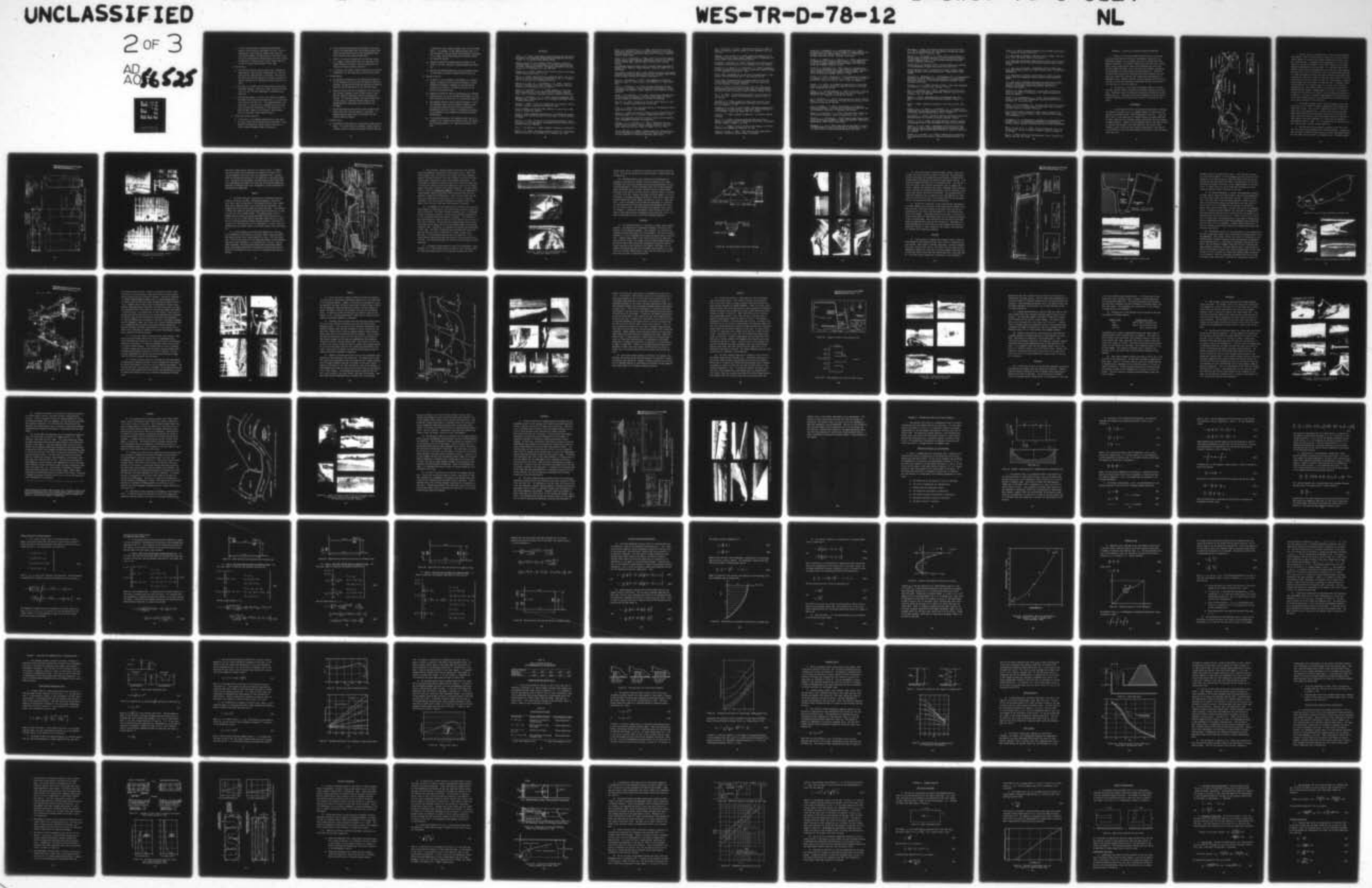

c. If the retaining dikes are constructed with borrow material from inside the containment facility, flow will concentrate in the resulting channels until they are filled with sediment, and the hydraulic efficiency of the area will be drastically reduced during the initial stages of operation. Periodic plugs or small finger dikes can be used to reduce channelization of this type.

4. Overflow Weir

a. The overflow structure should normally be a rectangular, sharp-crested, free-flow weir constructed in or near the dike with its length parallel to the dike. The slope of the dike below the weir should be as steep as possible to prevent resuspension of sediments from weir overflow currents.

b. The head, H, of water over the weir should be specified, preferably between 1 inch and 6 inches, and the unit flow rate, Q/L, over the weir determined in accordance with Figure 23.

c. The required length, L, of weir crest should be computed according to the influent discharge rate, Q_i, and the unit flow rate over the weir, Q/L. The computed length of the weir crest, L, may have to be increased by up to 10 percent to account for the effects of flow contraction. If the final weir length is not practical, then multiple weirs of equivalent length should be used.

5. Inflow and Outflow Locations

a. If the area is an elongated rectangle and no spur dikes are used, locate the inflow and outflow structures in the middle of the opposite, shorter sides of the area, or along a diagonal between corners if possible.

b. If the inflow and outflow must be located on the same side of the containment area, the distance between them should be maximized and one or more odd-numbered spur dikes should be constructed between them.

c. Either a long sharp-crested weir of sufficient length or a number of smaller weirs with the same total crest length should be utilized to prevent concentration of flow and increased approach velocities.

d. If possible, the general induced flow direction should be perpendicular to the direction of the prevailing winds.

6. Ponding Depths

a. The necessary information on the engineering properties of the foundation soils and the dike materials should be obtained, and appropriate stability analyses performed to determine the maximum height of the retaining dikes.

b. Based on density profiles of comparable sites and existing effluent standards, Figure 26 may be used to estimate the depth of withdrawal zone for a planned weir loading. For detailed designs, the reader should refer to "Weir Design to Maintain Effluent Quality from Dredged Material Containment Areas" by Walski and Schroeder.

c. The minimum ponding depth should be the estimated withdrawal zone plus a one-foot margin, if possible. If adequate ponding depth cannot be provided, the weir loading should be reduced.

7. Effective Area

a. The hydraulic efficiency of a planned area should be estimated. If no other data are available, assume square or irregular areas to be about 50 percent efficient and add

10 percent for each length-to-width ratio, M or M*, higher than 1. For example, if the length-to-width ratio is 4, add 30 percent to get 80 percent total hydraulic efficiency. In all cases, the estimated hydraulic efficiency should not exceed 90 percent.

b. Assume an operational ponding depth, and based on the slope of the site, determine a maximum ponded area possible.

c. Multiply the maximum ponded area by the estimated hydraulic efficiency to obtain a maximum effective area, A_e.

8. Retention Factor

a. To obtain a first estimate of the adequacy of a planned design, a retention factor may be calculated as follows. Although this method is adequate for the qualitative evaluation of a design, it should not be used in the design of containment areas.

b. Compute the retention factor by multiplying the estimated effective area, A_e, by the minimum ponding depth, D_p, and then dividing by the flow rate, Q, as outlined on page 43.

c. The computed retention factor should be between 1 and 10 and represents ½-day units of predicted retention time. The estimated retention factor can be qualitatively evaluated by the use of Figure 11 considering the nature of the sediments to be disposed and the desired removal effectiveness.

d. If considered necessary, the retention factor can be increased by (a) increasing the effective area, (b) increasing the ponding depth or (c) reducing the influent flow rate.

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APPENDIX A: REVIEW OF CE DISTRICTS DISPOSAL OPERATIONS

1. Districts with active disposal operations were visited and discussions were held with Planning, Engineering, Construction and Operations personnel on the problems and needs of present and future disposal operations. An informal interview was held with each relevant group, using a questionnaire specifically developed for this study. Following these interviews, one or more District personnel accompanied the investigator to inspect ongoing disposal operations. Samples of influents and effluents were collected from active disposal areas and were later analyzed to determine the concentration and gradation of suspended solids. Whenever possible, recent disposal operation reports, records, and plans were obtained to supplement the data collected during interviews or from field sampling.

2. The Districts visited were Philadelphia, Mobile, Galveston, Portland, Seattle, Norfolk, Baltimore, Charleston, Savannah, and Vicksburg. Additional reports and data were requested and received from the Detroit District. Following are narrative summaries of the discussions held and information collected for each District visited during this study.

Philadelphia

3. The Philadelphia District has extensive experience in dredging and disposal operations due to its mammoth project of maintaining navigation in the Delaware River "from Philadelphia to the Sea." Since the early 1950's, large quantities of dredged material have been deposited on land, primarily due to economic reasons (minimizing hauling distances of hopper dredges). However, the Philadelphia District has been involved in confined land disposal since 1880 (Mauriello and Caccese, 1963). Presently, over six million cubic yards are deposited on land annually at ten different sites along the Delaware and tributary rivers and canals (see Figure A1).

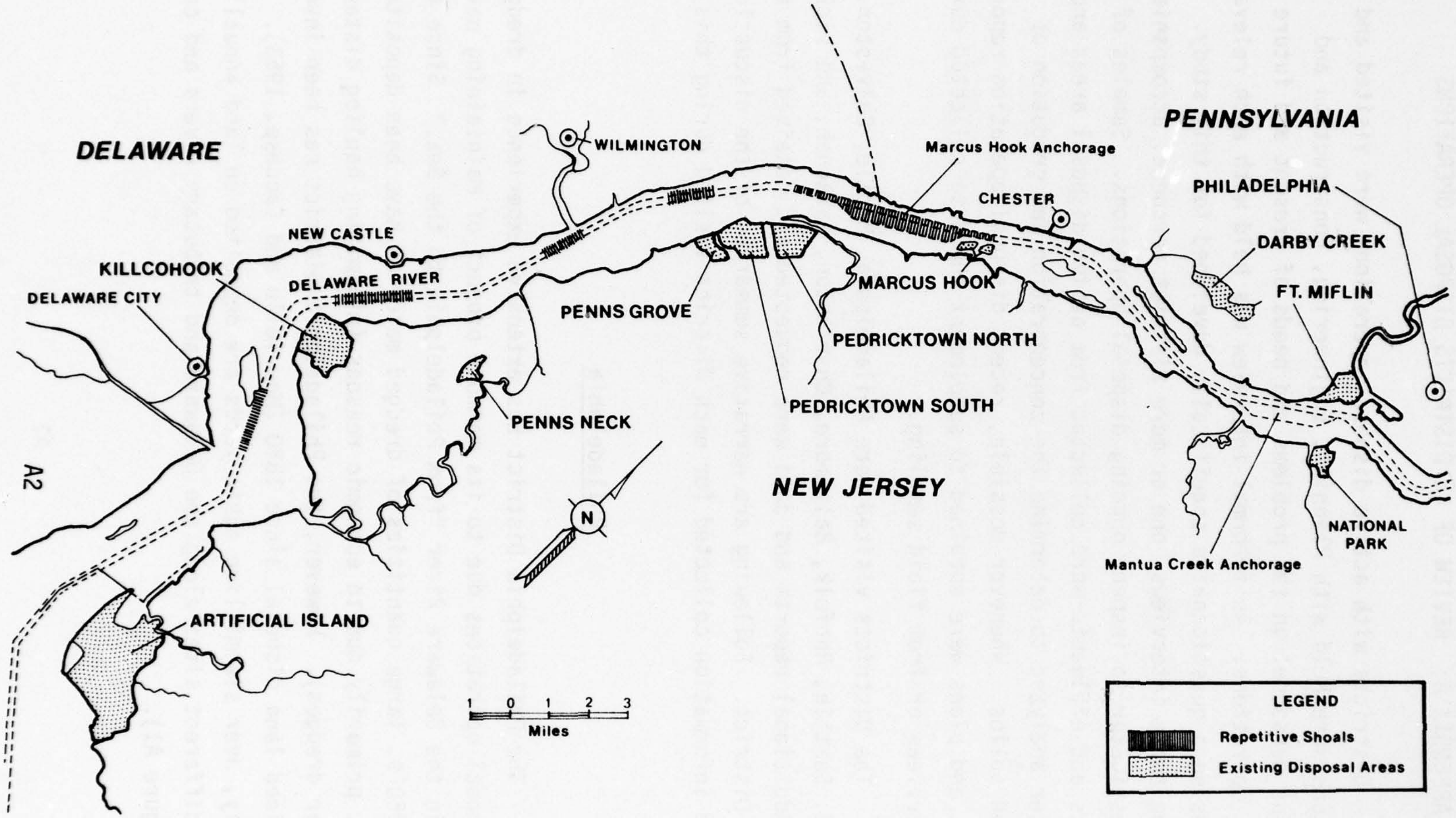

Figure A1. Philadelphia District dredging and disposal areas along Delaware River

4. Disposal sites are designed only to satisfy storage capacity requirements. Dikes are constructed of sandy soils, and very shallow ponding depths are used to avoid seepage through and structural failure of these dikes. A suspended solids concentration of 8 g/l above ambient is the quality standard for the final effluent and, although adequate documentation is not available, it is believed that the discharges usually contain suspended solids less than 2 g/l above ambient. New large (16 ft by 20 ft) steel weirs with a polygonal shape are used to replace old wooden box weirs which were destroyed during weed fires. A section of a new weir is shown in Figure A2, and pictures of both the old and the new weirs are shown in Figure A3. Most dredging is done by hopper dredge, but side casters are sometimes used. A complete dredging and disposal cycle requires about 2 to 3 hours, but the actual dredging period is only about 20 minutes long because no overflow is allowed from the dredges. Dredged material is mostly fine unconsolidated silts with densities of 1250 g/l and some sands with densities up to 2000 g/l. The cost of dredging ranges between \$0.75 and \$2.50 per cu yd, with disposal operations costing between \$0.15 and \$0.50 per cu yd or about 20 percent of the total cost.

5. The largest disposal site on land (Killcohook) covers 1400 acres, but it is divided into several compartments which undergo constant dike lifting. There is also a 3800-acre artificial island disposal site in the Delaware River. Disposal area selection has been based almost entirely on the proximity of the site to the shoaling areas, and several studies, including economic analyses of land and inland disposal, have been conducted (U. S. Army Corps of Engineers, Philadelphia District, 1969). It is estimated that the presently available sites provide a 20-year storage capability.

6. The dike construction generally conforms to the lay of the land and inlet-outlet arrangements are sited accordingly. Attempts are made to maximize retention times without utilizing large ponding depths, and spur dikes and cross dikes are frequently used. Confined dredged

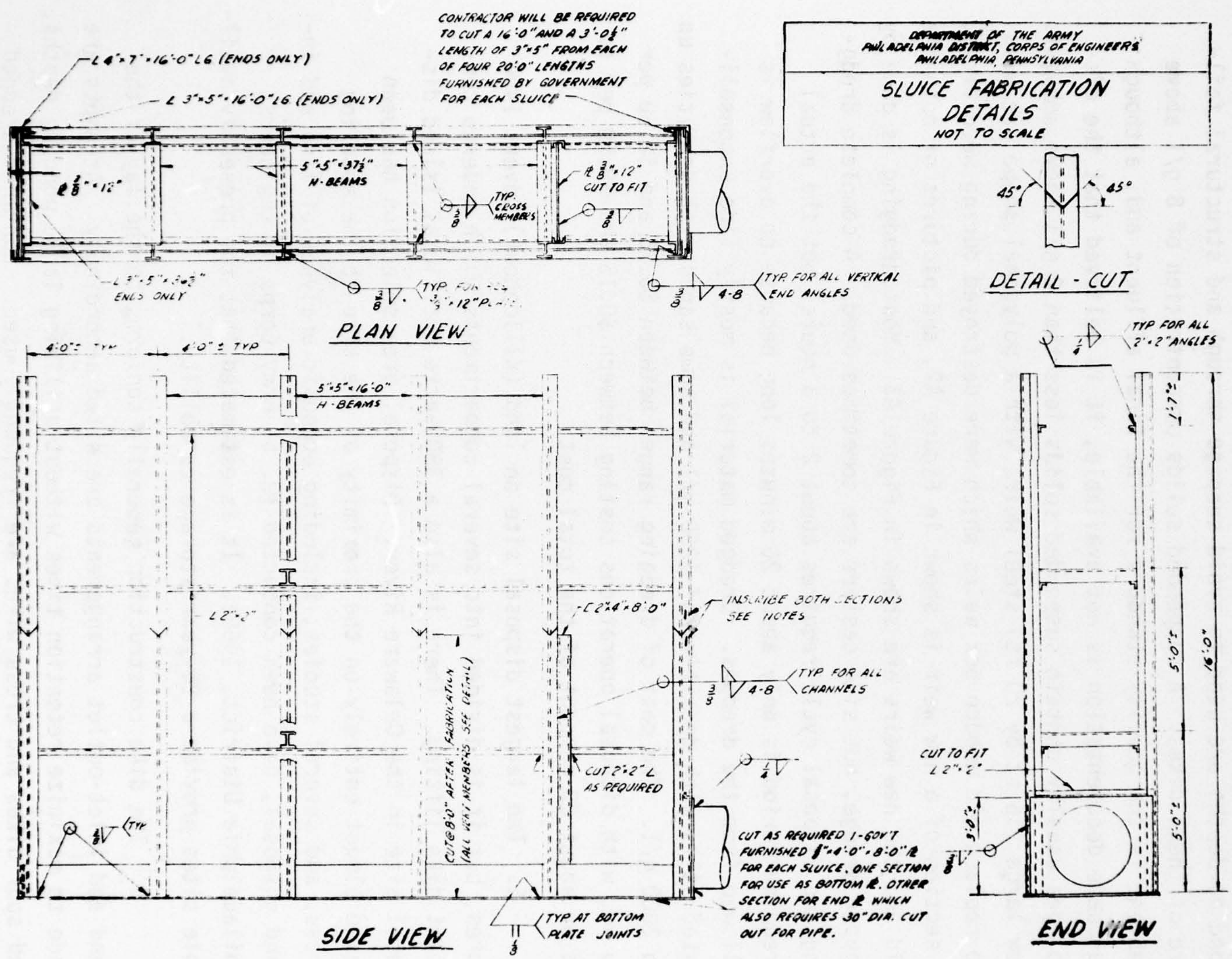

Figure A2. Details of Philadelphia District large weir section

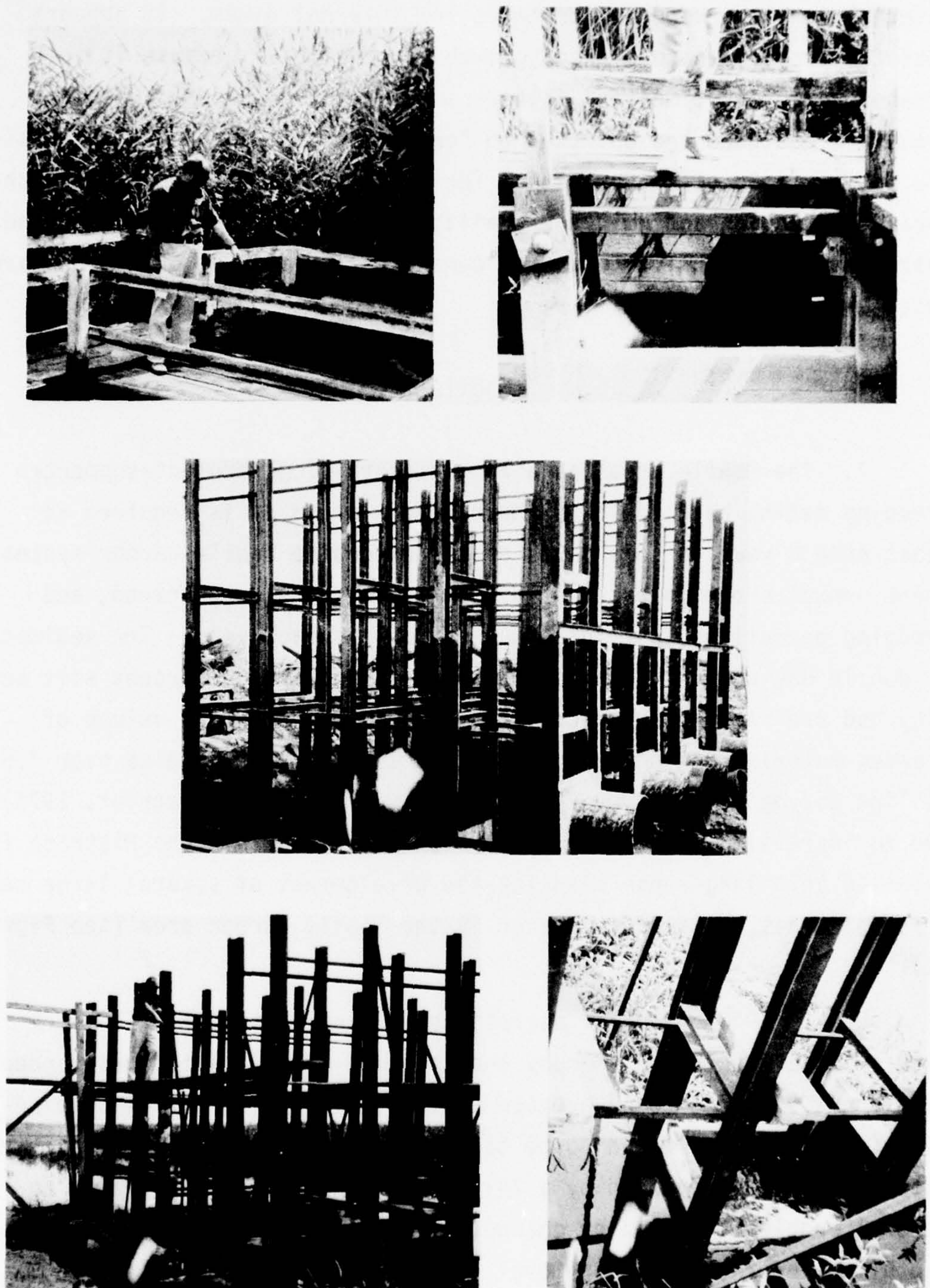

Figure A3. Old (top) and new (center, bottom) Philadelphia District weirs

material is often used to construct low internal dikes. It appears that the prolific natural weed growth is beneficial because it increases retention times and filters out suspended solids. Disposal areas are designed and operated to "get water out as quickly as possible." The field office personnel feel that there is very little technical information and guidance available for the efficient design and operation of disposal sites, and, consequently, current operations are based mostly on experience.

Mobile

7. The Mobile area has a long history of government-supported dredging dating back to 1826. Maintenance dredging is required at least once a year in some parts of the extensive Mobile harbor system where annual shoaling rates of up to two feet are encountered, and dredging operations last for about six months each year. The sediments in Mobile Bay consist of about 50 percent sand and 50 percent silt and clay and are rich in organic matter. The average annual volume of dredged material during the period from 1960 to 1970 totaled over 7.5 million cu yds (U. S. Army Corps of Engineers, Mobile District, 1975). Due to increasing pressures towards confined disposal, the District is involved in a long-range plan for the development of several large containment areas on islands located in the Mobile Harbor area (see Figure A4).

8. In 1970, disposal operations were conducted at a 130-acre site on McDuffie Island (Murphy and Zeigler, 1974). Water was ponded over about 90 acres to an average depth of 2.5 feet. A 27-inch pipeline ($\simeq$ 50 cfs) was found to be too large with respect to the discharge rate, and it was replaced by a 24-inch pipeline ($\simeq$ 40 cfs). Due to several problems, including channelization and insufficient ponding depth, the average solids removal efficiency of the area was only 75 percent, and effluents with up to 30 g/l of suspended solids were discharged.

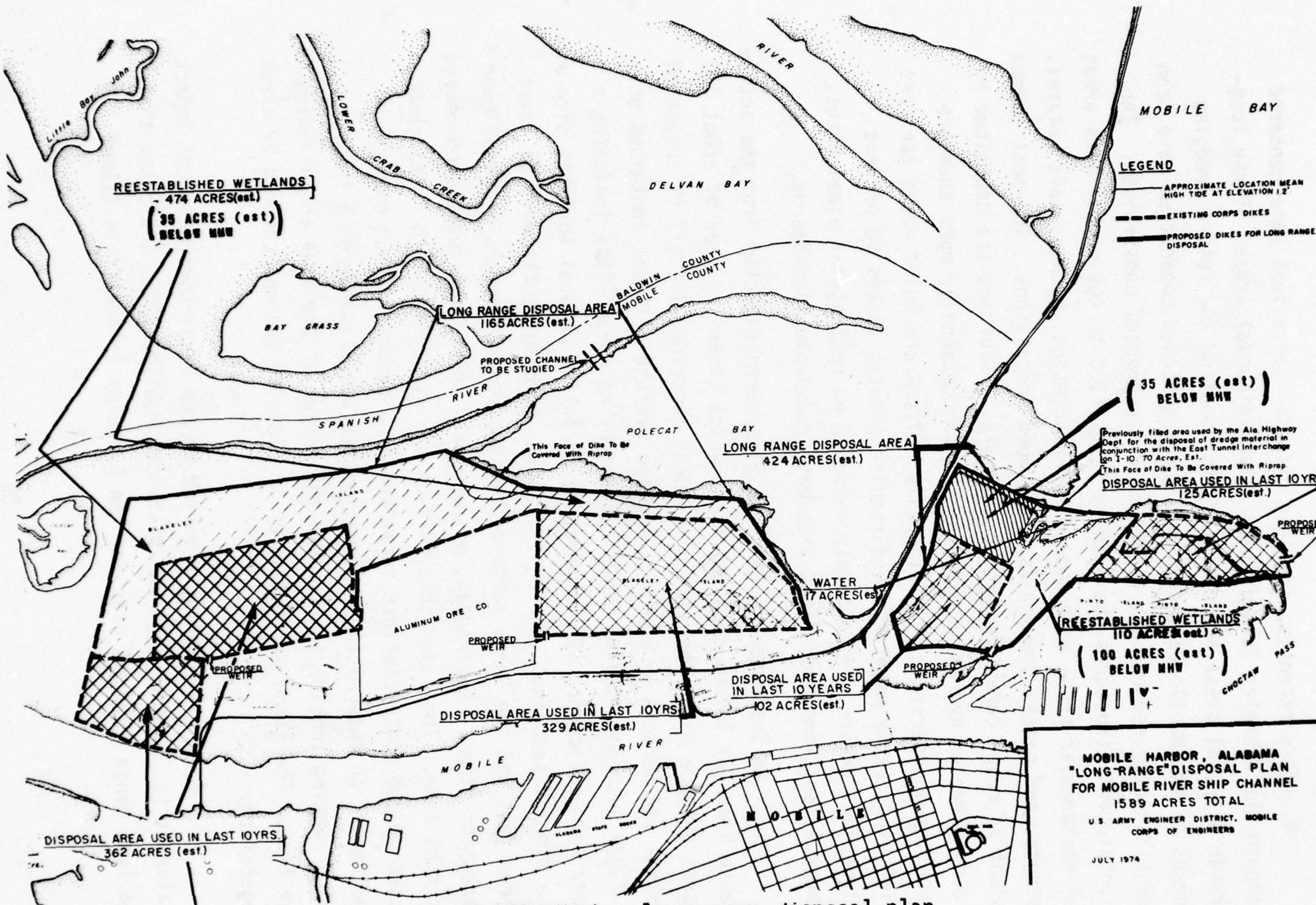

Figure A4. Mobile Harbor long-range disposal plan

9. More recently, confined disposal operations have centered around the Blakeley Island-Pinto Island disposal areas. These long-range disposal areas are essential for meeting the future dredging needs of the Mobile District, but their intended capacities have been significantly reduced due to strict environmental constraints. The containment structures are usually large (100 to 400 acres) open areas of rectangular shape with some marsh vegetation, such as salt grasses, sedges, and canes, growing as early weed successions. The most common plant is rosea cane (*Phragmites communis*). Slurries are deposited by large-diameter hydraulic pipelines, and occasionally more than one pipeline is operating simultaneously. Dikes are built about ten feet high using sandy materials and require plastic liners to prevent sloughing. Weirs are uniformly designed as rectangular steel boxes about 8 feet long and 4 feet wide with removable flashboards.

10. In September of 1976, samples were collected from the active South Blakeley Island disposal site, which covers an area of about 300 acres (Figure A5). The slurry influent contained 132 g/l of suspended solids (12 percent by weight), and the average effluent contained only 0.2 g/l suspended solids (0.02 percent by weight). This indicates a very high efficiency (99.85 percent) of solids removal for this site at the time of sampling. During that time, ponding depths were at least three feet and the large surface area and ponding depths provided good retention times. Apparently, no resuspension problems were encountered at that time. The major through-flow was stratified with quite clear water on top and a distinct interface at a depth of about one to two feet. An interesting observation was the presence of a low ridge running longitudinally down the center of the area and dividing the flow. The ridge was formed by an old dike and was covered by marsh vegetation.

11. An economic study reported in the Final Environmental Impact Statement (U. S. Army Engineer District, Mobile, 1975) indicates that the long-range disposal plan for the Blakeley Island-Pinto Island

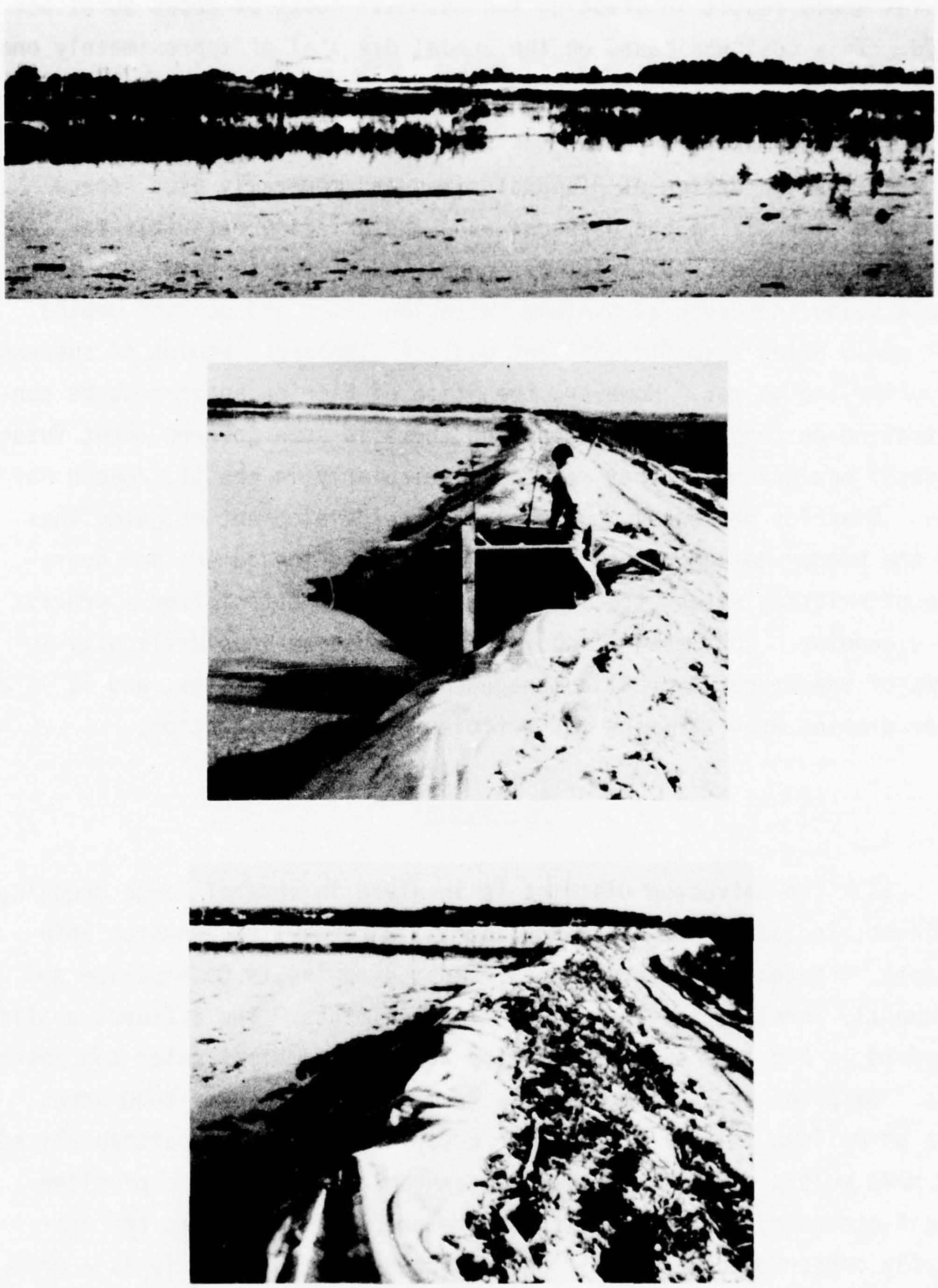

Figure A5. Photos of the South Blakeley Island disposal area, Mobile District

complex would result in dredging and disposal costs of about $0.81 per cu yd. This cost was based on the annual disposal of approximately one million cu yd.

12. The Mobile District has received little information or guidance for the design of disposal sites and generally uses "space available" for sizing the disposal area and existing materials for constructing the dikes. District personnel would like to have available simple guidelines such as minimum retention times and ponding depths that would allow specific effluent quality standards (amount of suspended solids) to be met. However, the State of Florida appears to be concentrating on turbidity standards and there is some concern about future disposal operations in that region, particularly in the St. Joseph Bay area. District personnel would welcome the development of guidelines for the proper design of new sites and the good management and operation of existing sites, the latter being more important from a practical viewpoint. Contractual requirements create serious difficulty in terms of the proper hydraulic management of existing sites, and it is a major problem when striving for efficient disposal operations.

Galveston

13. The Galveston District is involved in several large dredging projects, including the Gulf Intracoastal Waterway, the Houston Ship Channel, and the Galveston Channel. Most dredging is by pipeline and frequently involves the removal of fine sediments. The effluent quality standard is 8 g/l of suspended solids above the ambient water concentration. Disposal areas range in size from 50 acres to over 1000 acres, such as Pelican Island. The larger areas are usually compartmentalized and have multiple inlet and outlet arrangements to minimize pipeline length according to the location of the dredge. Spur dikes are frequently constructed near the final sluiceway, which normally is a drop inlet box structure located about 30 feet from the dike inside the disposal area, as shown in Figures A6 and A7.

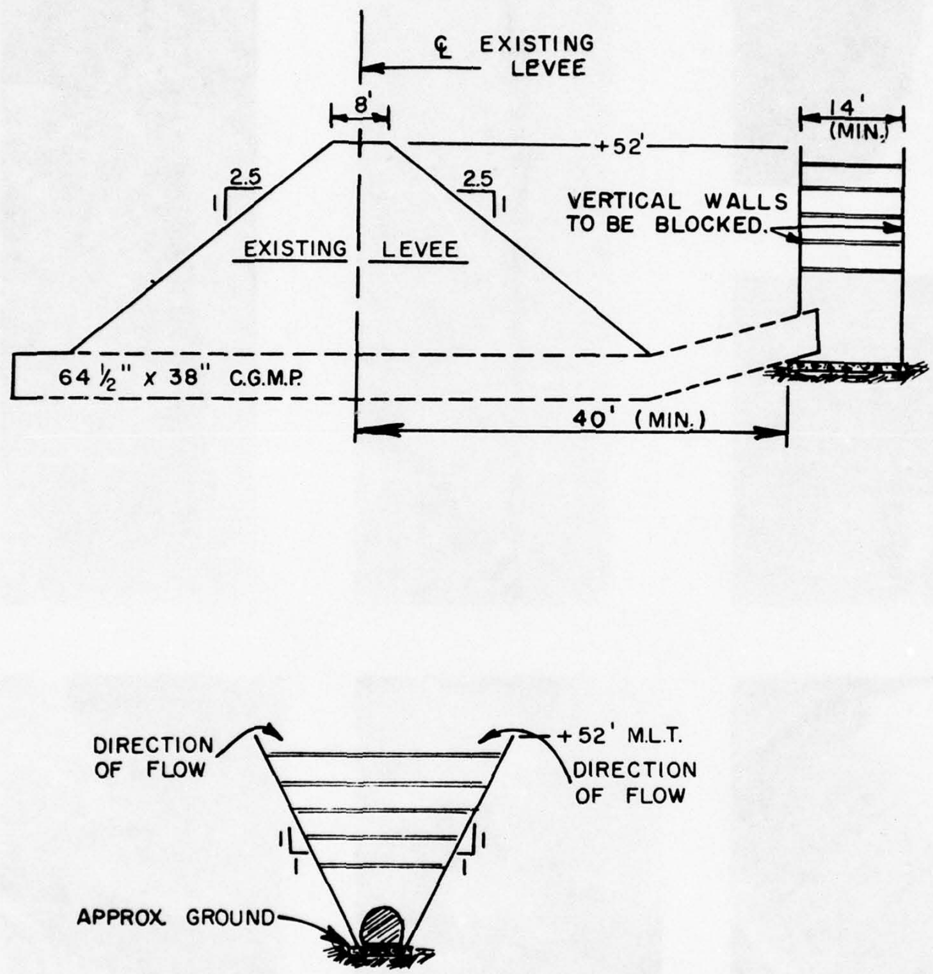

Figure A6. Galveston District drop inlet spillway

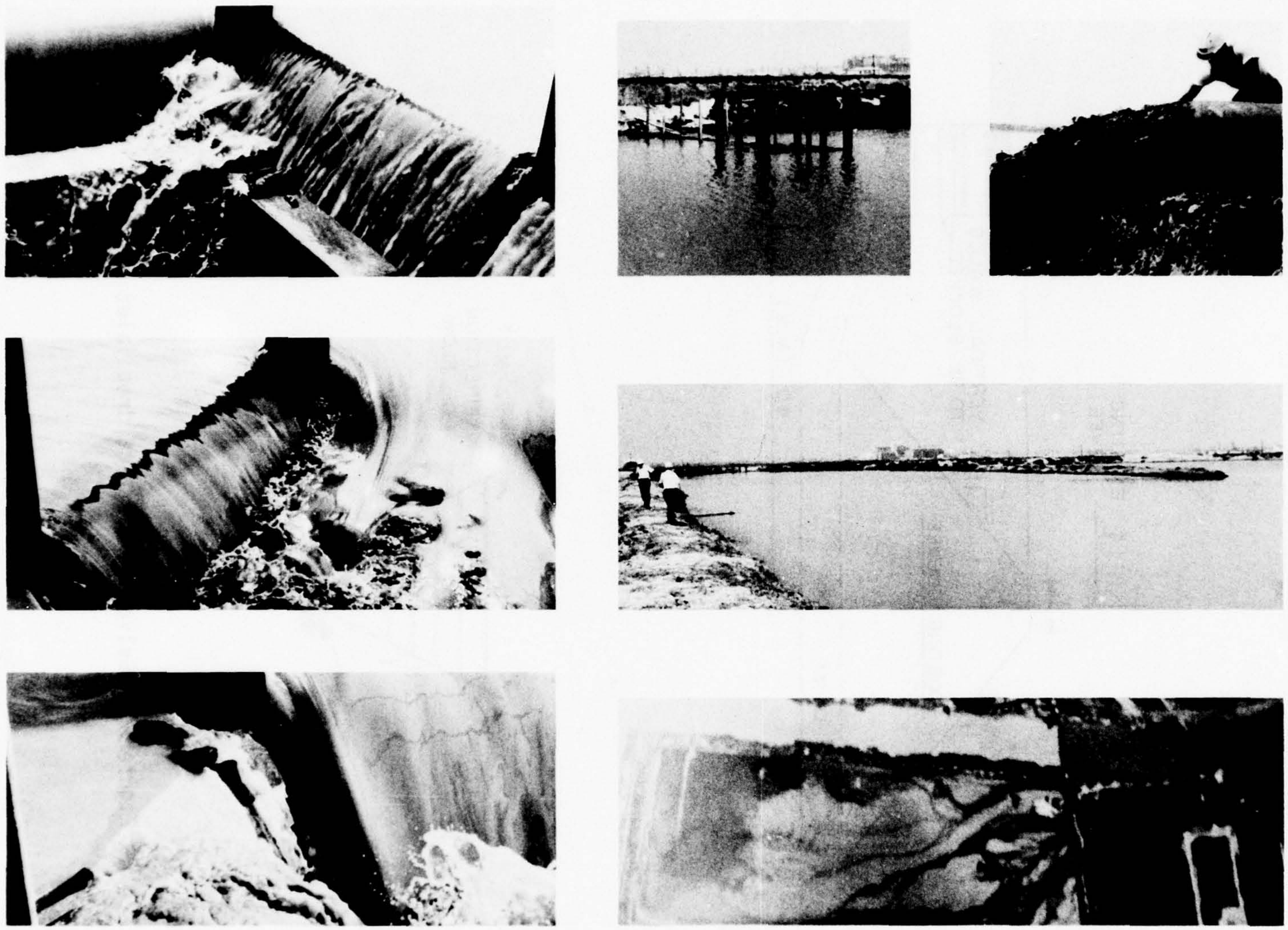

Figure A7. Disposal areas 2 and 3, Freeport, Texas

14. The active disposal areas at Freeport Harbor, Texas, were visited, and their outlines are shown in Figure A8. The performance of these large open areas seemed to be significantly affected by the strong prevailing winds and the effluents from Disposal Area No. 2 contained an average of about 15 g/l of suspended solids. The area has a size of 50 acres, and a 24-inch pipeline was used for pumping the slurry. However, the influent slurry had a suspended solids concentration of over 300 g/l (26 percent by weight), and this indicates a relatively high removal efficiency of 95 percent. The extremely dense influent was apparently due to pumping of very fine material at a high rate (15 ft/sec or more), but it is unlikely that this was representative of the average conditions usually encountered.

15. During site inspection, it was suspected that resuspension of bottom sediments was occurring in the narrow channel leading to the box inlet weir (Figure A8) because of increased velocities due to a combination of wind effects and the reduction in the cross-sectional area of the channel. Stratified flow with clear water on top was observed in the middle of the disposal area, but the turbidity increased as the flow approached. Both horizontally and vertically stratified water was observed going over the weir (Figure A7). Resuspension of bottom sediment due to wind effects and high through-flow velocity is explained in detail in other sections of this report, and guidelines for avoiding this problem are advanced.

Portland

16. A wide variety of dredging and disposal projects are being administered by the Portland District office, which is faced with unique problems. One project at Chinook, Washington, involved a disposal area with a size of only 3 acres. This small area was made available immediately adjacent to a boat basin that was dredged (Figure A9), and 20-foot-high dikes (Figure A10) were constructed with a front end loader. An 8-inch cutter dredge with a discharge of 6 cfs resulted in

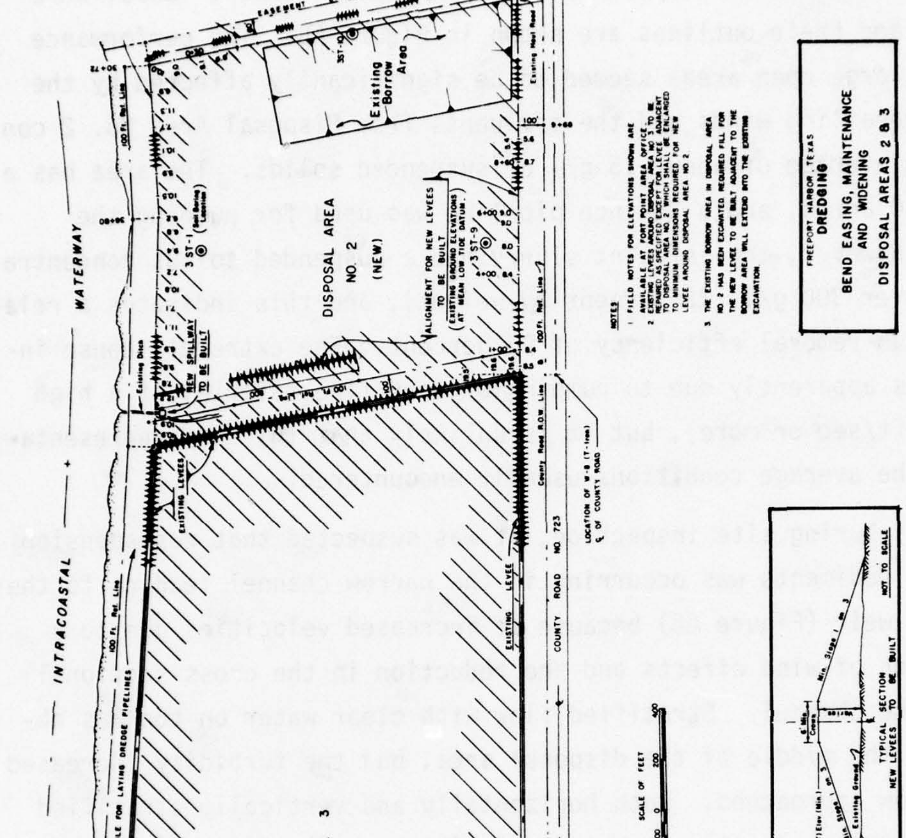

Figure A8. Plan of disposal areas 2 and 3, Freeport, Texas

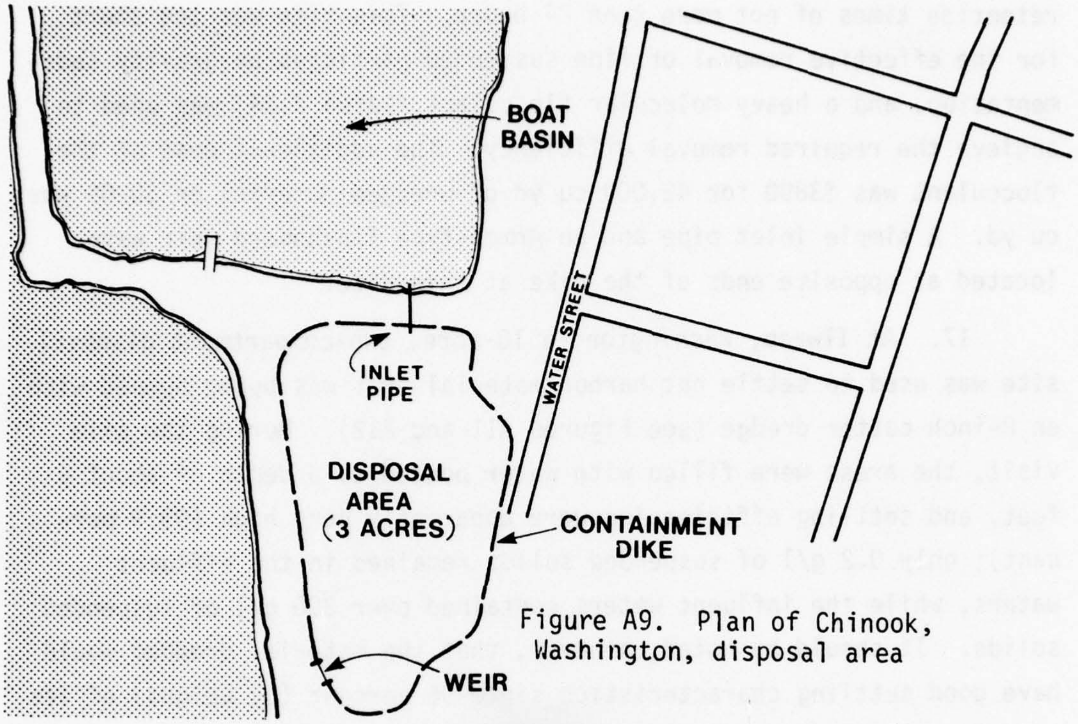

Figure A9. Plan of Chinook, Washington, disposal area

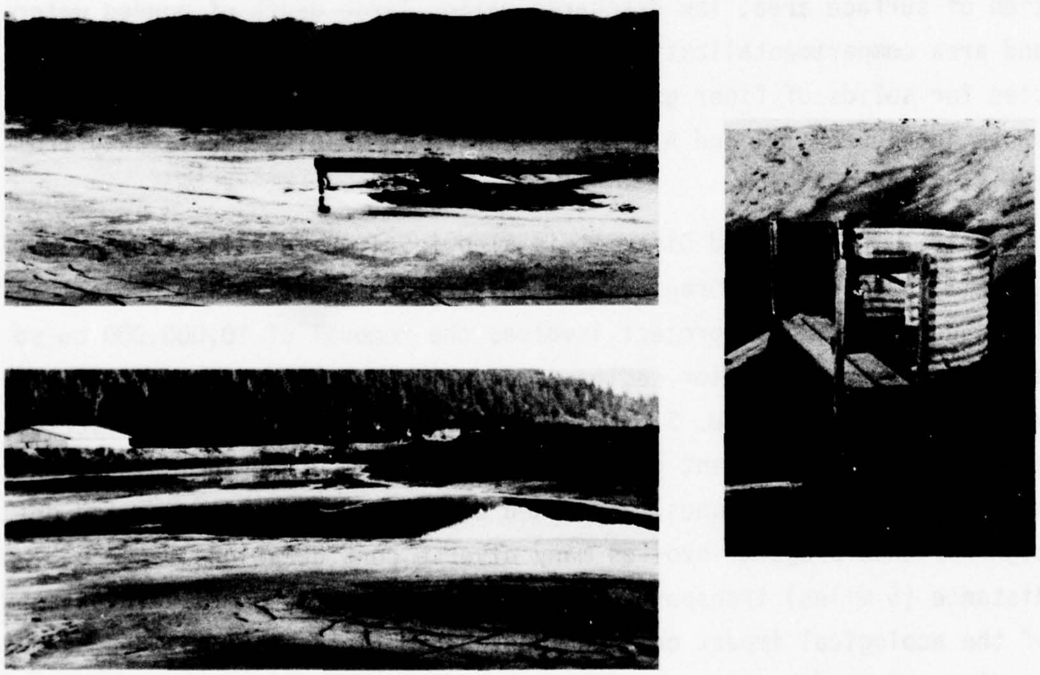

Figure A10. Photos of Chinook disposal area

retention times of not more than 24 hours. This time was too short for the effective removal of fine suspended particles by gravity sedimentation, and a heavy molecular flocculant (Nalco 7134) was used to achieve the required removal efficiency. The additional cost of the flocculant was $3800 for 42,000 cu yd of dredged material or $0.09 per cu yd. A simple inlet pipe and an Armco-type flashboard weir were located at opposite ends of the dike at this site.

17. At Ilwaco, Washington, a 10-acre, two-compartment disposal site was used to settle out harbor material that was being pumped with an 8-inch cutter dredge (see Figures A11 and A12). During the site visit, the areas were filled with water ponded to a depth of about 6 feet, and settling efficiencies were apparently very high (99.9 percent); only 0.2 g/l of suspended solids remained in the effluent waters, while the influent waters contained over 300 g/l of suspended solids. It should be noted, however, that the material dredged would have good settling characteristics since 96 percent (by weight) of its particles were coarser than 10 microns. However, the existing combination of surface area, low discharge ratio, large depth of ponded water, and area compartmentalization could provide adequate removal efficiencies for solids of finer gradation. A simple floating log barrier, as shown in Figures A11 and A12 was effective in keeping debris away from the final sluice pipe.

18. The Portland District is also involved in a large program (Figure A13) at Coos Bay, Oregon, where a deep-draft navigation project is being undertaken. This project involves the removal of 10,000,000 cu yd of different types of bottom sediments of which approximately 8,000,000 cu yd are sand and silt (U. S. Army Corps of Engineers, Portland District, 1975). Over 25 different disposal sites will be used, including open ocean, beaches, sand dunes, and diked pasture lands. Planning and design for this project involves many diverse considerations, such as long-distance (5 miles) transport of slurry using booster pumps, evaluation of the ecological impact on sand dunes and wetlands that will be used as disposal areas, and special provisions for revegetating several of

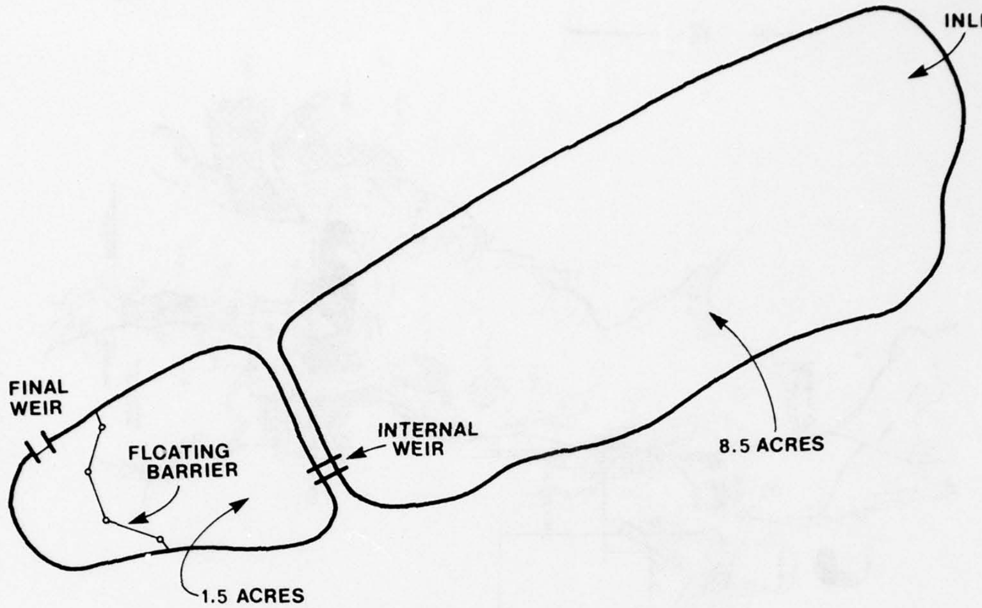

Figure A11. Plan of Ilwaco, Washington, disposal area

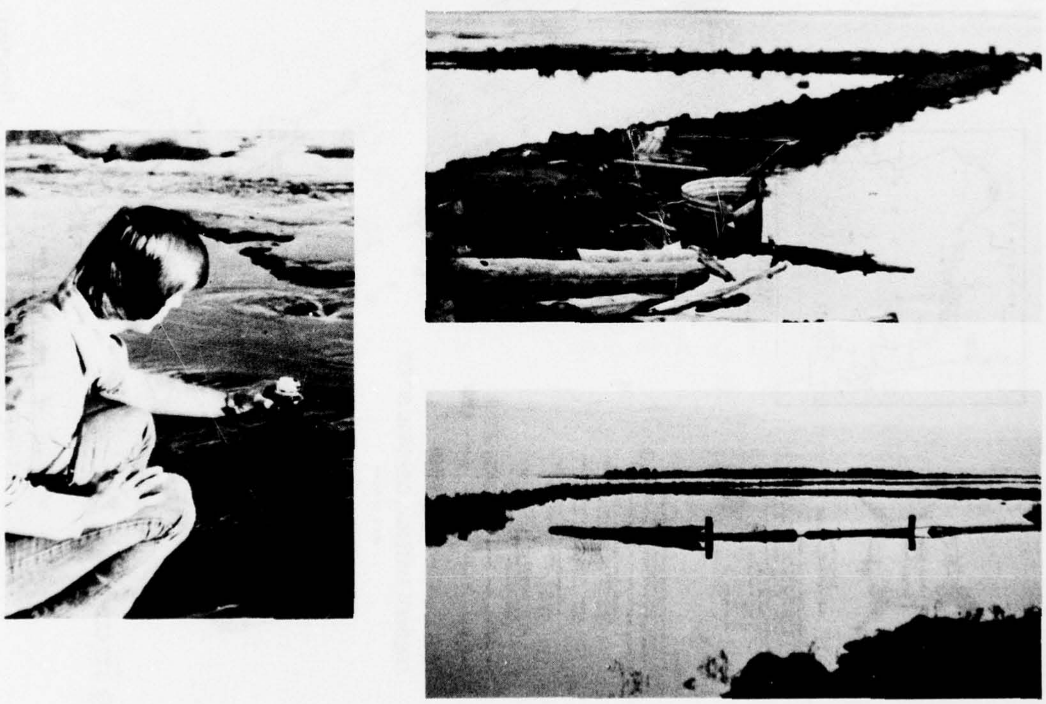

Figure A12. Photos of Ilwaco, Washington, disposal area

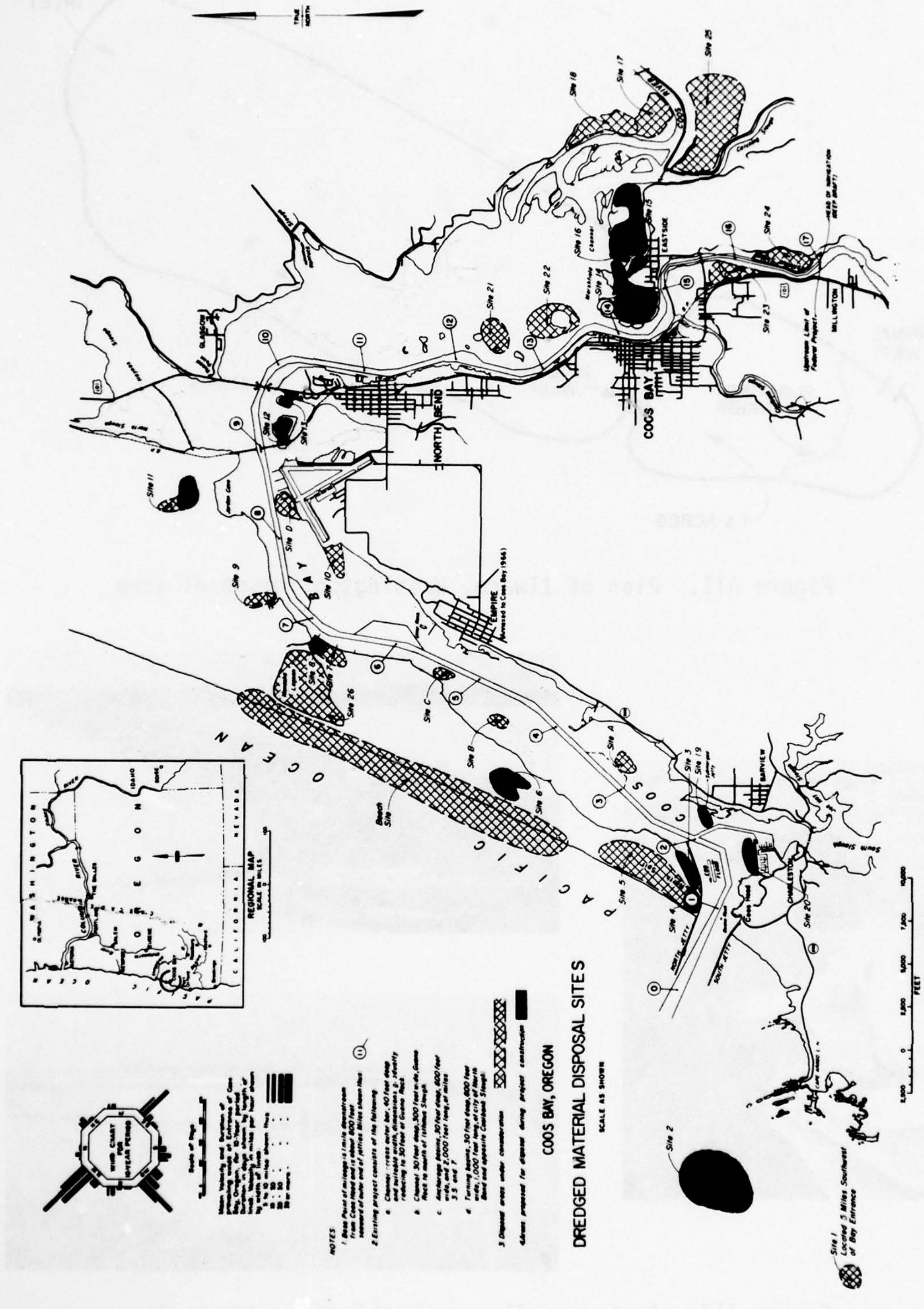

Figure A13. Coos Bay, Oregon, dredging and disposal plan

the affected sand dune areas. Recent cost estimates indicate about $1.70 per cu yd for operations requiring 5 miles of booster pumping, and $1.15 per cu yd for 1.5 miles of pumping. This may be construed to reflect separate booster pumping costs of approximately $0.10 to $0.15 per cu yd per mile over a base dredging cost of about $1.00 per cu yd. However, this is not an accurate assessment of actual costs, because long-range booster pumps have very high fixed costs; for example, the mobilization costs alone for the Coos Bay Project were almost 3 million dollars. The average cost for this project is $1.15 per cu yd and a unit value of $0.15 per cu yd can be conservatively allocated for direct costs of dike and spillway construction and maintenance. In addition, $900,000 was allocated for the total, non-federal disposal costs of easements, dikes, spoil areas, etc., and an estimated $100,000 for revegetating the sand dunes inundated by disposal operations. Based on the above estimates, approximately 15 to 20 percent of the total cost of sand and silt dredging will be for land disposal requirements, exclusive of booster pumping.

19. The Coos Bay Project involves several large disposal sites, such as the Christiansen Ranch area, which is compartmentalized. Figure A14 shows the construction of a huge weir which is oriented perpendicular to the dike. A 3-inch-maximum head of water over the weir is used for design guidance. The Portland District uses a turbidity criterion to gauge the settling effectiveness, and each contract has a shut-down clause based on compliance with this criterion. The contractual requirement for the new Coos Bay Project specifies an upper limit of 50 JTU for disposal area effluents.

20. Finally, District personnel felt that the available methods of designing and operating disposal areas are adequate for the present, but may not be suitable in the future due to changing requirements. The District uses grain size analyses of sediments for disposal planning and would like to be provided with guidelines for the determination of ponding depth with respect to the type of material dredged and the dredge size.

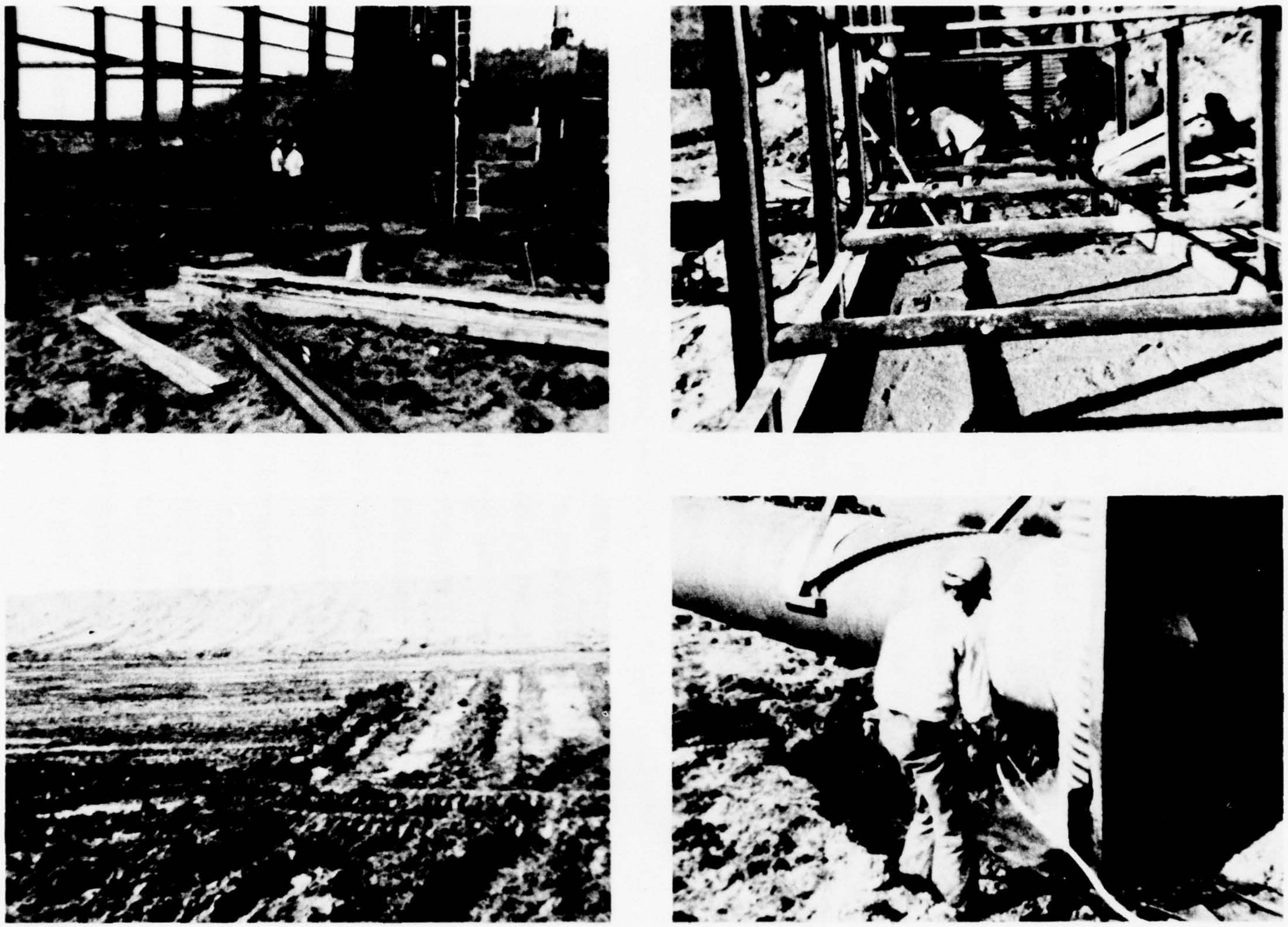

Figure A14. Photos of Coos Bay disposal area, Portland District

Seattle

21. Due to the area's extensive fishery and wildlife resources, this District has encountered considerable opposition to and regulation of its dredging projects. Almost all projects are regulated on a case-by-case basis with many different agencies and interest groups involved. The Environmental Protection Agency has imposed a unique standard of 50 ml/l settleable solids on the effluents from most of the disposal areas. In addition, extensive biological and chemical tests are conducted in situ, including monitoring of caged fish at sensitive marine resource areas where dredging is conducted.

22. Although several projects were discussed and observed during this site visit, the most informative case was the Willapa Harbor dredging project. This unusual site was originally constructed as two separate disposal areas and then connected in series (Figures A15 and A16) with a channel and an auxiliary pumping scheme so that effluents would comply with strictly enforced quality standards. The first pond had been filled and was inactive at the time of the site inspection (the channel between the two ponds was abandoned and the dredge was discharging directly into the second pond). Due to the unavailability of land, this site was simply too small to handle the discharge generated by the 20-inch dredge used for this project and unfortunately very little could be done to improve the settling effectiveness, except to impose intermittent shutdowns as necessary.

23. Samples collected during this visit are not considered representative of the prevailing site conditions. The dredge had shut down but, due to poor adjustment of the final overflow weir, severe resuspension of bottom sediments resulted in effluents of very poor quality. Samples of effluents at the final overflow weir were not collected; instead samples were taken just after the first weir that was discharging into a final raceway. The suspended solids were unusually high (36 g/l), which reflects a very poor removal efficiency (75 percent). However, when the weir malfunction was corrected, the final effluent

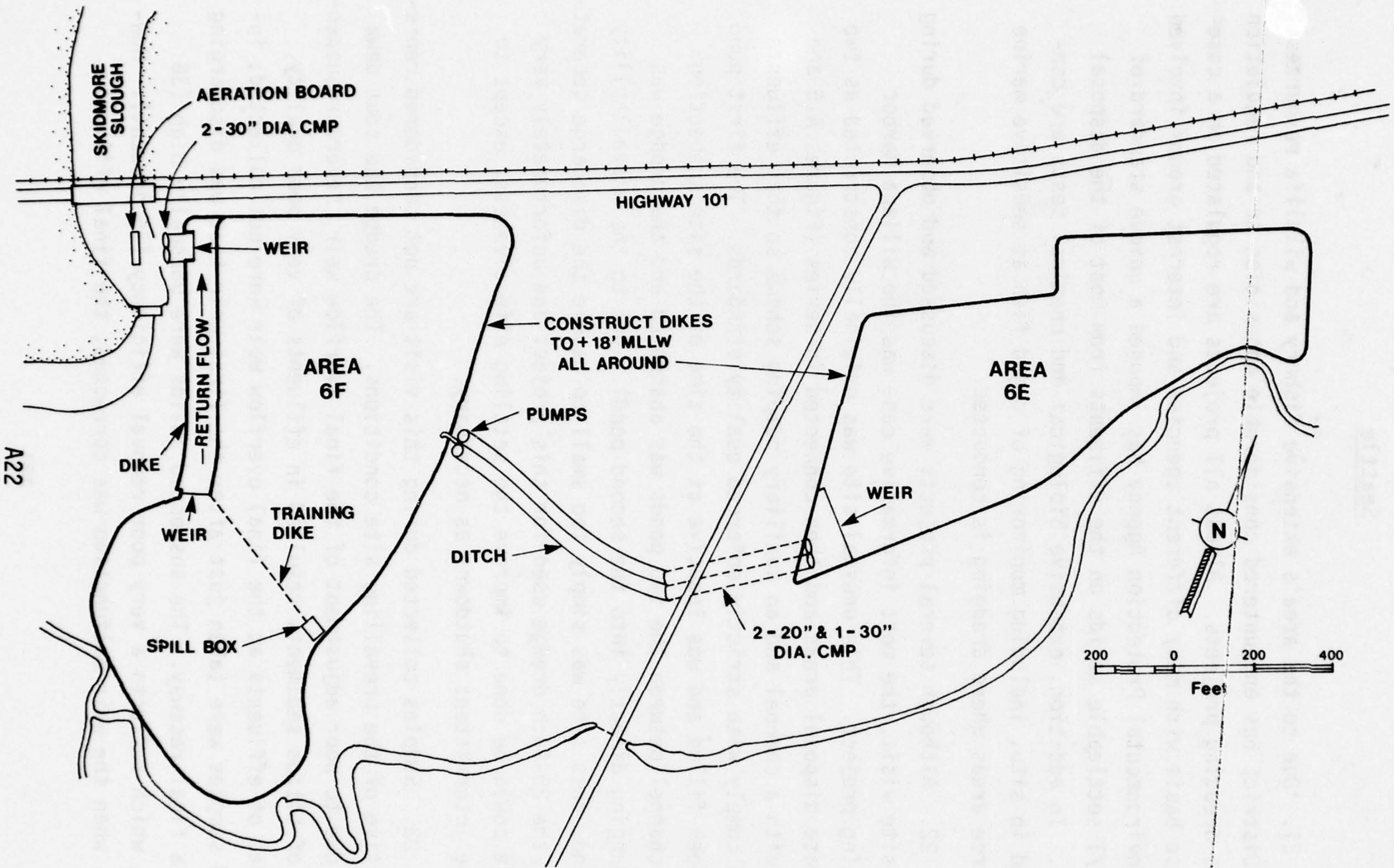

Figure A15. Plan of disposal areas 6E and 6F at Willapa Harbor, Washington

Figure A16. Photos of disposal area 6F at Willapa Harbor, Washington

samples had a very low concentration of suspended solids (less than 1 g/l). Nevertheless, these samples were also not representative, because the discharge of the slurry had been discontinued and the confined waters were drawn down, resulting in long retention times. A reasonable estimate is that normal continuous operations at this site would produce effluents with an average suspended solids concentration of about 18 g/l. This level of performance is considered consistent with disposal sites which have a very small effective area and receive slurries from a 20-inch dredge. In addition, the rather close proximity of the inflow pipe to the overflow weir may have caused short-circuiting in the area. Attempts are often made to avoid short-circuiting in the areas designed by the Seattle District office by channelizing the flow and using internal berms to help retain material. Weirs are usually designed to maintain not more than 4 inches of head over the crest and, at the Willapa site, the weirs were rather small, simple box structures. Larger disposal areas have been provided with very large, square drop inlet weirs which are located away from dikes inside the disposal area. A unique feature of Seattle District disposal areas is a weir of fixed crest elevation which is constructed of horizontal boards and is located just upstream of a spillway or a sluicing device as shown in Figure A16. Finally, the effluent discharge pipes which terminate in tidal rivers and sloughs are fitted with sea valves or hinged covers to prevent backflow into the disposal area during high tides.

24. Very little design guidance is available for disposal site planning, and uniform design practices would be very helpful in improving disposal area operations. However, uniform guidelines may be difficult to implement because of the extensive involvement of many agencies and private groups in the disposal planning processes.

Norfolk

25. The Norfolk District is best known for its large disposal area called Craney Island. This in-water structure was originally built in the early 1950's to reduce disposal costs by minimizing the ocean hauling distances of dredged sediments. Three sand levees, each about two miles long, were constructed in shallow waters using hydraulic pumping techniques, and an area with a size of about 2500 acres was enclosed, as shown in Figure A17. About 120 million cu yd of material have already been stored in this site, and only a few more years of operation are possible, unless the dikes are raised. The presently authorized fill height is 18 feet above mean low tide. Although slurry quantities of up to a million cu yd per month are sometimes pumped into this site, heavy rainfalls are often of greater quantity and create more concern about the structural integrity of the area. At Craney Island, one weir is rectangular with a length of 24 feet and a width of about 5 feet. Two other weirs at this site are E-shaped with dimensions as shown in Figure A18. External slope damage from hurricanes and other storms was a serious problem until adequate dike protection was accomplished with the use of riprap. Original cost for this disposal site was over $6,000,000 and annual maintenance costs are about $250,000. Storage space alone costs about $0.07/cu yd and other costs for rehandling are assessed if required. The latest dredging costs were about $0.80/cu yd for 1,650,000 cu yd of material.

26. Two other disposal areas are of interest in this District: the Hoskins Creek and the Yorktown sites. The Hoskins Creek area is a multi-level diked enclosure necessitated by dike stability problems. Borrow pits or trenches just inside the dike perimeter tend to channelize waters, as shown in Figure A19, and the filtering ability of the interior vegetation is lost. To avoid this problem, horizontal plugs are left in the trenches every few hundred feet, but channelization still occurs after the low areas have been filled with dredged material. The Yorktown site had a similar short-circuiting problem which was

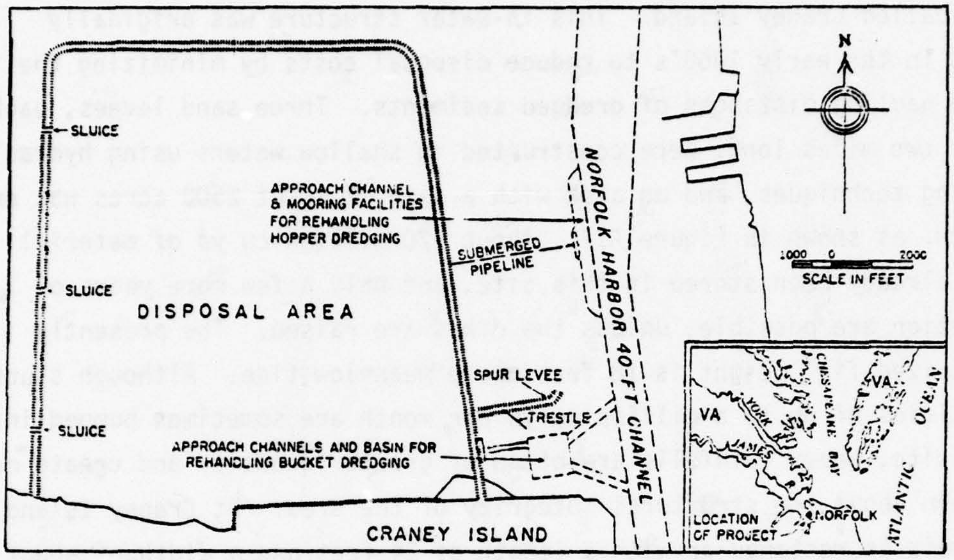

Figure A17. Layout of Craney Island disposal area

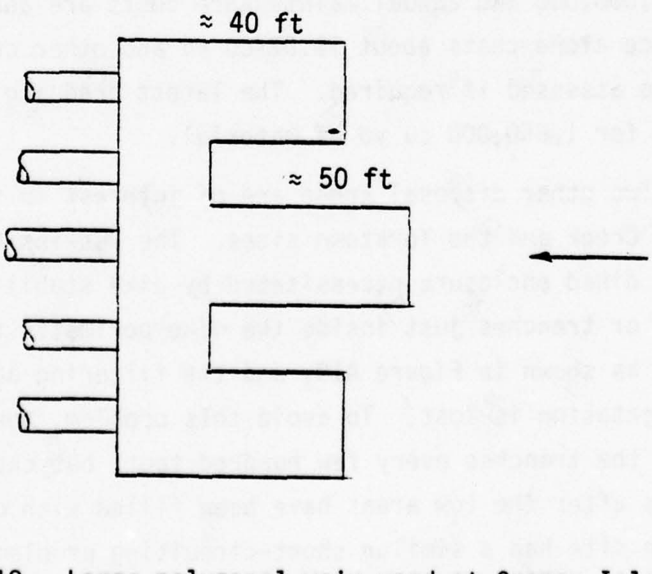

Figure A18. Large polygonal weir used at Craney Island

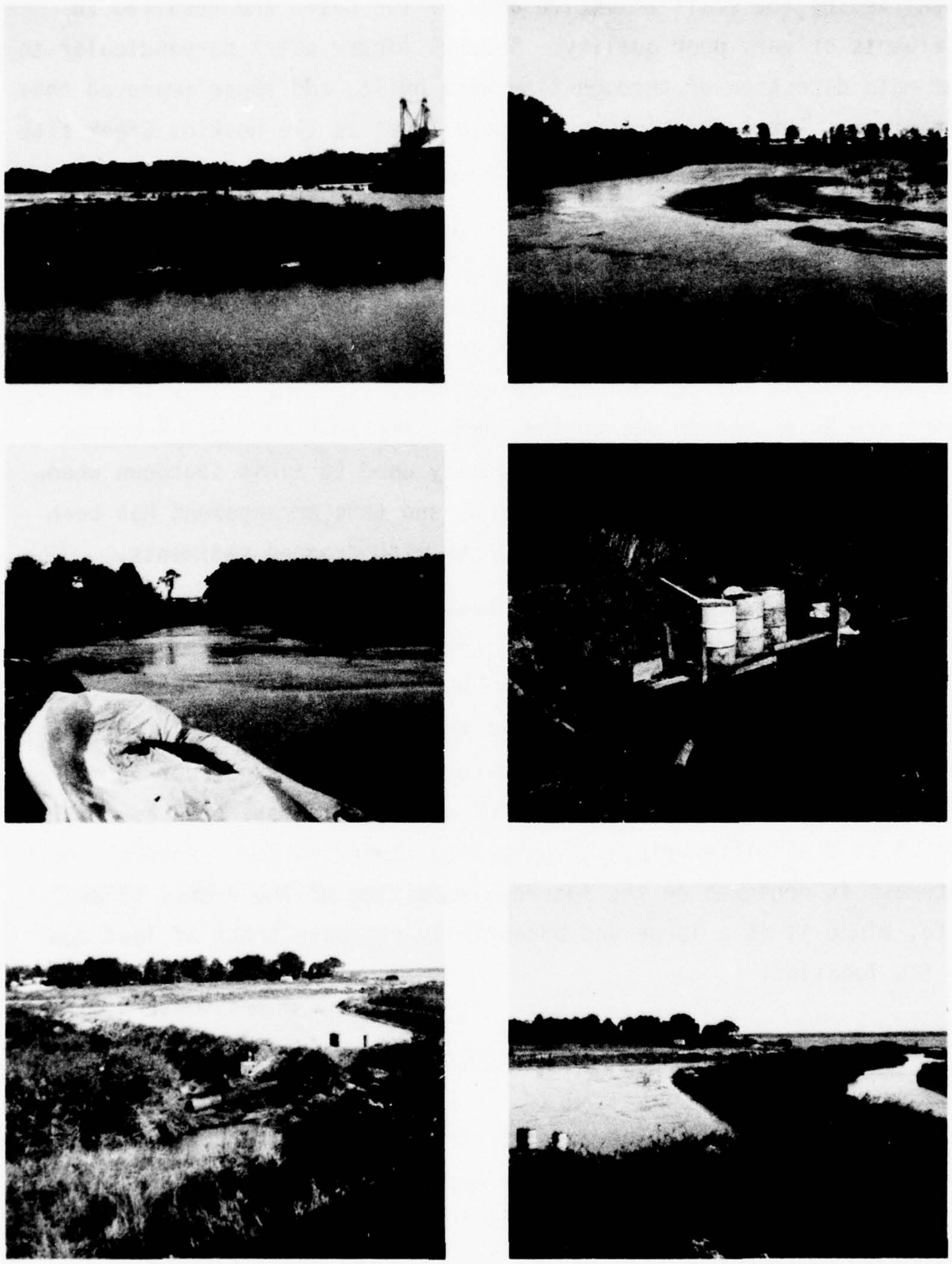

Figure A19. Photos of Hoskins Creek disposal area, Norfolk District

magnified by the small effective area of the basin and resulted in effluents of very poor quality. Several finger dikes perpendicular to the main direction of through-flow were built, and these improved this situation. Total dredging and disposal cost at the Hoskins Creek site for almost 100,000 cu yd was $0.97/cu yd, of which $0.34/cu yd was for dike construction.

27. Prefabricated steel weirs, usually 8 feet by 8 feet in size, with a square drop inlet structure have been used for years in this District. The total weir length is generally 14 feet for 12-inch or smaller dredges and 28 feet for larger dredges. The Craney Island weirs are an exception due to the large size of the site. Y-connections on influent pipes are occasionally used to avoid shutdown when increasing the length of the pipeline, and this arrangement has been observed to help fill in corner pockets with dredged sediments.

28. Disposal area design is based entirely on experience and economics because no written guidelines or specifications are available. Each project is handled on a case-by-case basis. Local interests are frequently responsible for supplying "suitable" disposal areas which must provide adequate material for the construction of dikes. It is generally felt that the average disposal area is only useful as a wildlife refuge or park after being filled. However, much interest is centered on the future disposition of the Craney Island site, since it is a large and potentially valuable tract of land due to its location.

Baltimore

29. In this District most of the decisions regarding construction and operation of disposal areas are made by the contractor. However, dikes are usually limited to 10 feet in height, unless otherwise approved by the Corps of Engineers. The effluent quality standard restricts suspended solids concentrations in the effluents to less than

13 g/l above ambient water concentrations. If effluent qualities approach this limit, additional water is ponded or pumping operations are discontinued temporarily. The contractor monitors the effluents and the Corps of Engineers and the Environmental Protection Agency conduct spot checks. The suspended solid concentration in the effluent is measured by use of hydrometers.

30. Guidelines for sizing disposal areas are based on the type and quantity of dredged material:

Material Type	Disposal Area Sizing
Sand	1300 to 1400 cu yds/acre-foot
General	800 to 900 cu yds/acre-foot
Fine Silt	600 cu yds/acre-foot

Considerable use is made of compartmentalization and finger dikes at several disposal areas to improve basin performance. Weir design is left entirely to the contractors, who frequently construct weirs with three concrete wall sides, each about four feet in length, providing a total weir length of 12 feet. No specific guidelines concerning weir length or crest heights are available. Most dredging projects utilize 12-inch dredges. The location of influent pipes is based on convenience, and Y's are often used to facilitate the extension of the pipeline.

31. Three recent dredging projects had costs of $1.07, $1.15, and $1.82 per cu yd, respectively. No data were available on separate disposal costs. There is a distinct trend to confine all dredged material in disposal areas, regardless of its pollution potential, in order to simplify operations. Present disposal operations are considered very satisfactory and most of the responsibility is passed on to the contractor. The only problem is obtaining suitable disposal areas from local interests.

Charleston

32. Many disposal sites have been acquired through easements along river banks. All disposal sites are in marshy areas with extensive vegetation. Usually, low dikes (5 to 6 feet high) are initially constructed, and then are subsequently built up with the dredged material, which is typically a grayish clay called Cooper marl. The sites are operated by keeping water levels as low as possible and redistributing the material throughout the areas as much as possible to prevent mounding. The Charleston District has quite a few years of storage area available, and District personnel would like to see better guidance on the management of existing areas to improve their performance.

33. The 500-acre Yellow House Creek site (Figure A20) was visited during active disposal operations. The flow seemed to be channelized and deep water and high velocities were observed at the weirs, which were set in pairs forming a "Y" spillway, as shown in Figure A20. Standard 6-foot-wide Armco-type weirs are used, but consideration is now given to the use of aluminum weirs. The inlet pipe is equipped with an adjustable deflector, as shown in Figure A20, which can be set to control flow patterns and better distribute incoming material. The samples collected at this site indicate good removal efficiencies (about 93 percent at the first weir grouping and 96 percent at the second weir grouping which is at a greater distance from the inlet pipe). The heavy vegetation would normally provide even better removal efficiencies for the 18-inch dredge being used, but the forced channelization necessary for the redistribution of the clayey material might be causing resuspension. This is a good example of the trade-off dilemmas that many Districts face. Deep ponded water and low velocities facilitate effective settling but create threats to the integrity of the dikes, and the necessary channelization for material distribution can also induce bottom scouring and resuspension.

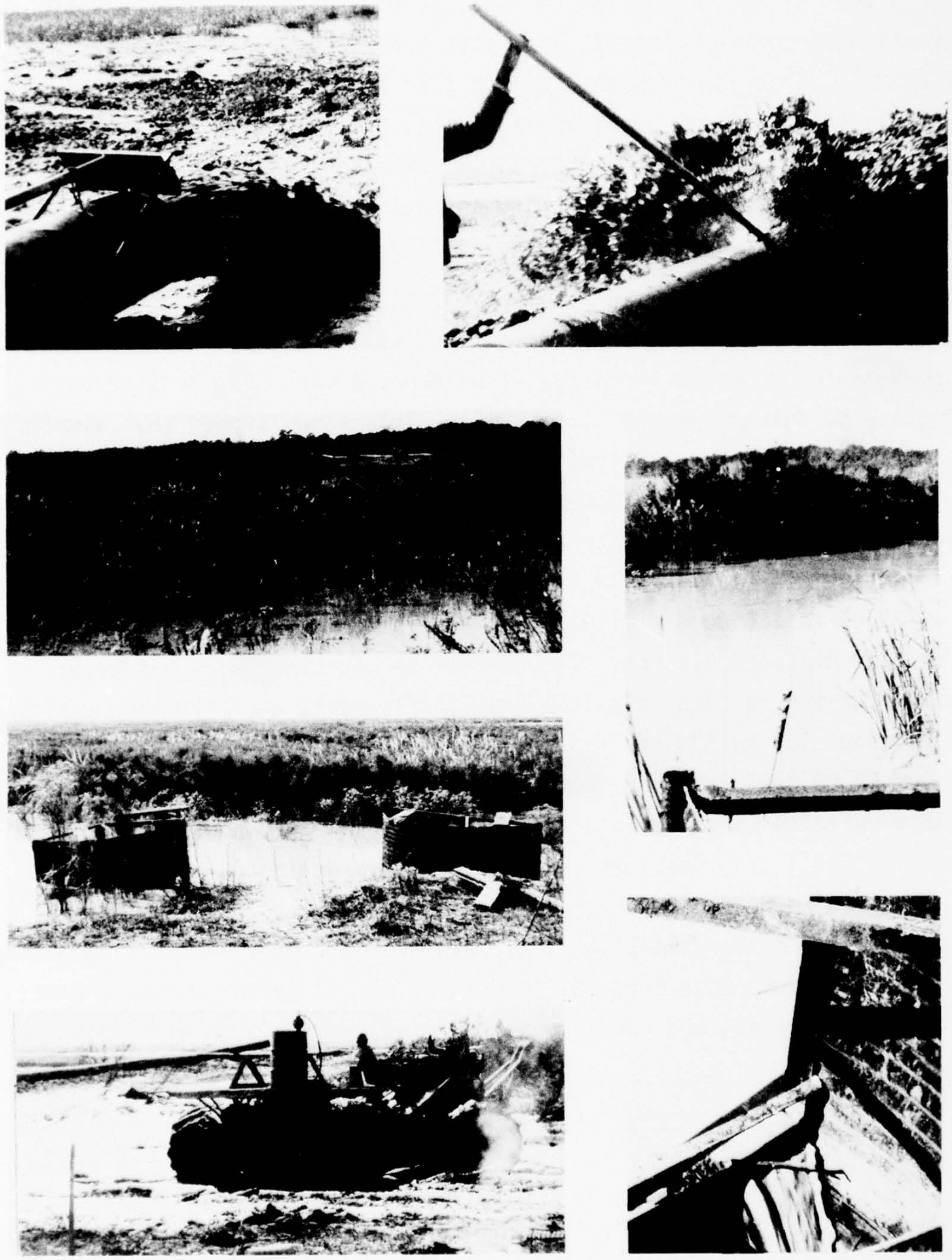

Figure A20. Photos of Yellow House Creek disposal area at Charleston, S. C.

34. No specific guidance is available for disposal area design or effluent quality control, and water quality is monitored only visually. District personnel do not feel present methods for sizing and planning disposal areas are adequate, but they must work with whatever sites are available. The redistribution of material and the necessity for keeping areas as dry as possible are the major problems, and considerable interest was expressed in the Riverine Utility Craft (RUC)*.

35. During 1966, the Charleston District conducted an extensive study of future needs for disposal of dredged material (U. S. Army Engineers, Charleston District, 1966). This study showed that simple low dikes were being constructed for about $2.00/ft in the 1950's and $4.00/ft in the 1960's. Some more elaborate dikes would cost up to $8.00/ft while drainage ditches were estimated to cost about $1.00/ft. Current prices (1977) would probably be double the 1960 estimates; that is, $8.00 to $16.00 per linear foot for low dikes and $2.00 per linear foot for drainage ditches. The same study discussed many different plans for future total disposal area development, and estimated costs were about $22 to $28 per foot of dike. This would be equivalent to about $2000 per acre of disposal site, depending on its size and shape. These estimates included clearing of areas, spillway construction, engineering and legal fees, etc., and resulted in estimates of disposal costs from $0.02 to $0.06 per cu yd of shoal volume. Again, these costs would be approximately double at present (1977) prices, indicating a disposal cost range from $0.04 to $0.12 per cu yd for the same particular requirements and conditions.

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*The Riverine Utility Craft (RUC) has been used in disposal areas in the Mobile and Charleston Districts to perform various tasks, including dewatering and densification so that conventional support equipment can operate in the area (Willoughby, in preparation)

Savannah

36. The Savannah District operates several very large disposal areas, such as the Barnswell Island area No. 13, which is 2000 acres in size and is shown in Figure A21. While flowing through these large areas, dredged material slurries are effectively clarified by the extensive filtering effect provided by the abundant vegetation (mainly Spartina grass) and internal dikes are not necessary. Redistribution of material to avoid mounding is necessary and the RUC was tested in this area and found to be very effective for creating redistribution channels. A channel recently constructed by the RUC is shown in the photos in Figure A22. The Savannah District personnel believe that this experimental device could possibly be improved by attaching a plowing mechanism behind it to form channels in the hard layer of sediments that exists below the soft surface layer.

37. Ponding up to 1000 acres for mosquito control has been tried, but no ponding is preferred to minimize the danger of dike failures. The large disposal areas have many multiple weirs and inlet pipe arrangements for operational flexibility. Several weir locations were selected after a dike failure had occurred, and the weirs were constructed in the location of each breach because these were obviously the areas where waters tended to pond and needed to be released. The standard Armco weirs are now being replaced by new aluminum weirs with two sets of 3-foot-wide flashboards for ease of handling. In disposal area No. 13, six weirs having a total crest length of 36 feet are located at various points. The cost of a single weir equipped with a 36-inch effluent discharge pipe is $2000. The average cost of dike construction is estimated to range from $0.75 to $1.00 per cu yd of material used, while dredging costs are very low, ranging from $0.19 to $0.34 per cu yd.

38. Dike design varies according to the nature of the foundation soils. If dikes are built too high (20 to 25 feet) in a continuous operation, they may settle 3 to 6 feet in a very short period of time.

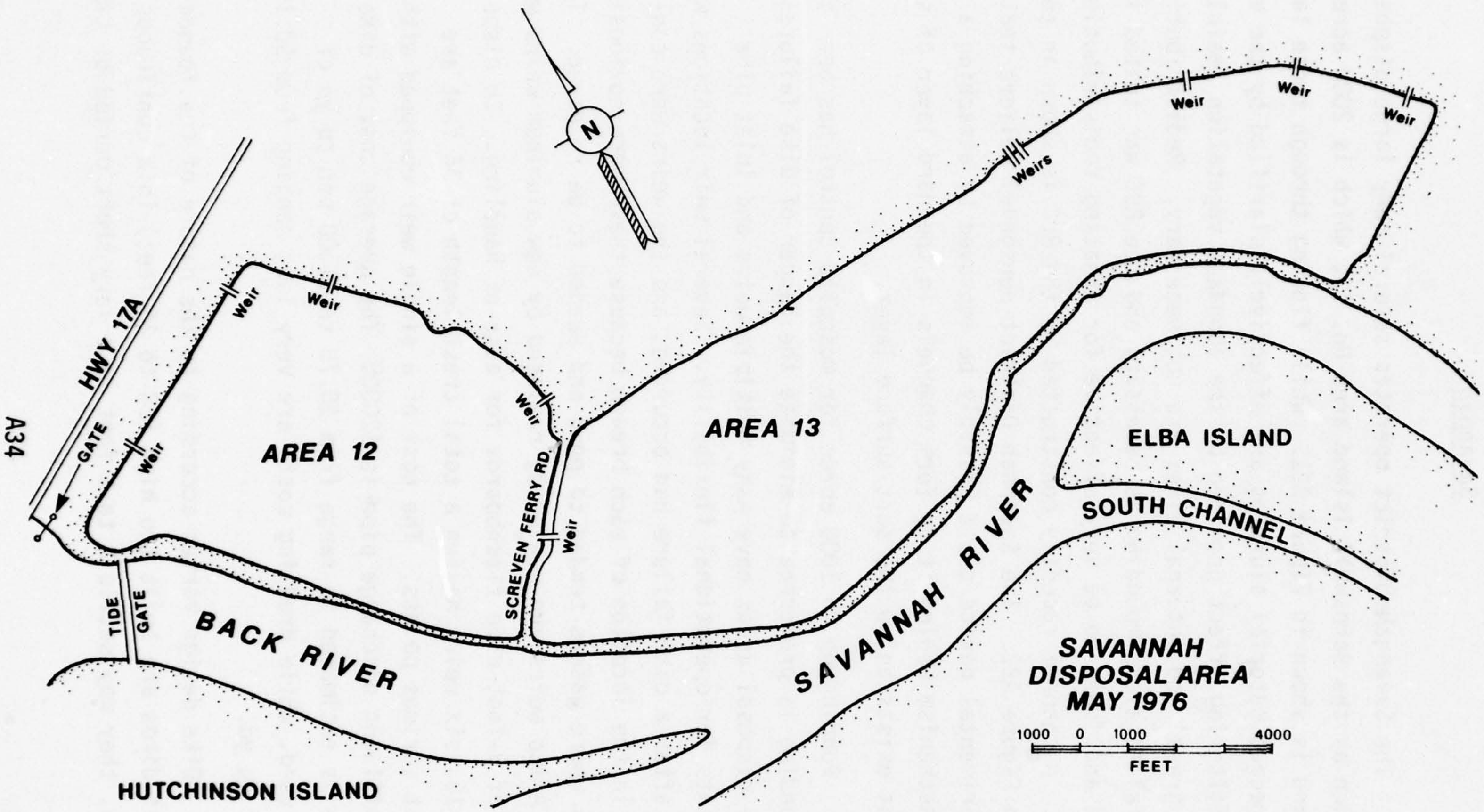


Figure A21. Plan of disposal areas 12 and 13 at Savannah, Georgia

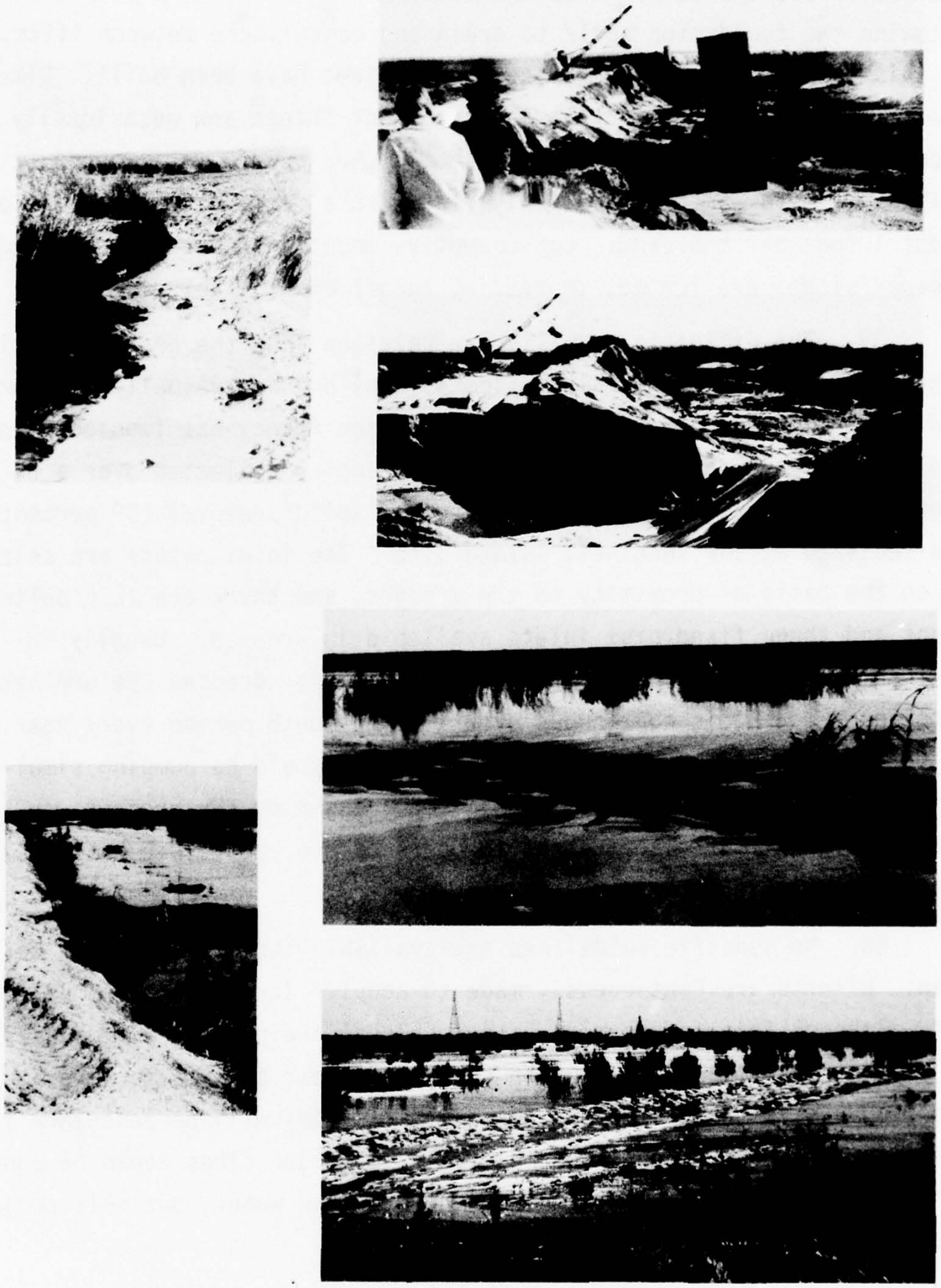


Figure A22. Photos of disposal areas 12 and 13 at Savannah, Georgia, showing dike repair and weir emplacement (top) and redistribution channels (bottom)

The best procedure is to raise the dikes gradually (once a year), allowing the foundation soils to drain and consolidate between lifts. In this manner, dikes as high as 30 to 35 feet have been built. Dike breaching in these large areas is a constant threat and occasionally occurs, especially during heavy rainfalls when runoff water ponds quickly. The areas are relatively flat with a maximum ground slope of about 1 foot per thousand. Consequently, mounding occurs quickly and steeper slopes are formed, increasing runoff considerably.

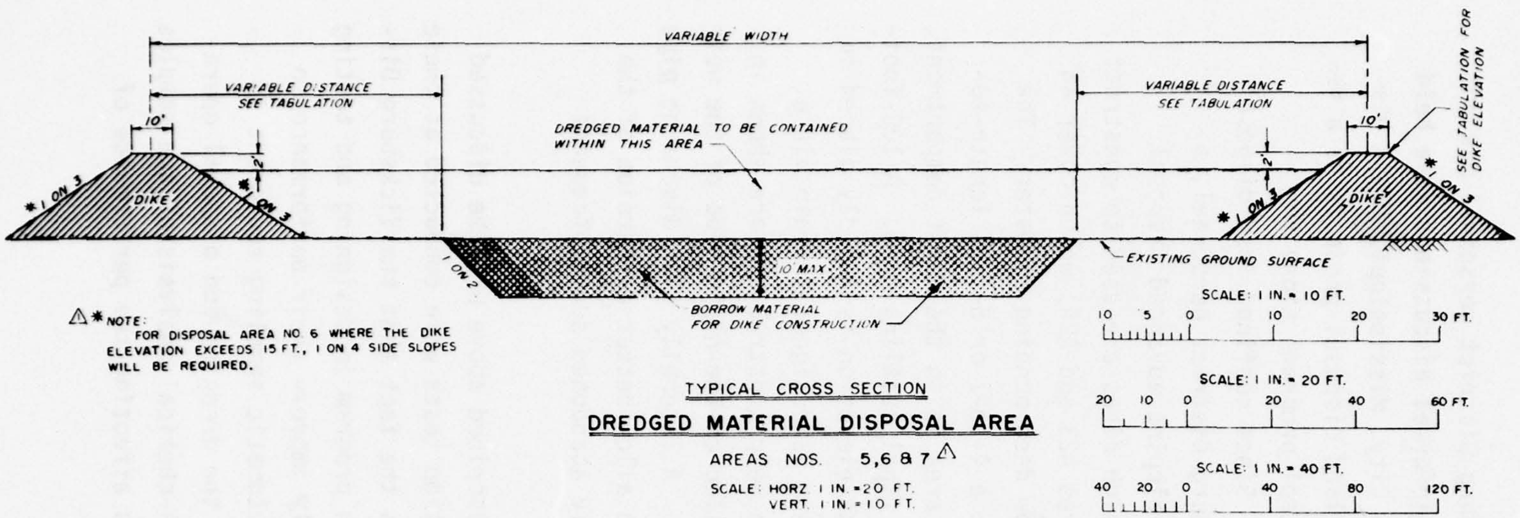
39. The effluents normally are released into the nearby brackish waters (due to environmental considerations) and are visually monitored and controlled. The Environmental Protection Agency has imposed a control criterion of 50 JTU in some cases. Samples collected over a 6-month period show that very good suspended solids removal (99 percent) was realized at the Barnswell Island Site. The inlet points are selected on the basis of proximity to the dredges, and there are 15 pipeline ramps and three fixed pipe inlets available in area 13. Usually 18- to 24-inch dredges are used, but occasionally larger dredges are employed. Pipeline dredging is continuous over a seven-month period every year. At times, one, two, or three separate dredges could be pumping simultaneously into the Barnswell Island area. Based on the present use rate, it is estimated that about 10 years of storage volume wa still available at this site.

40. No specific guidelines are available for disposal area design. Efforts are continuously made to acquire large tracts of land but, although this arrangement is possible at the present time (1977), it may not be a feasible solution within the next ten years. The impression exists that different disposal practices will be necessary in the future. Engineering personnel felt that wider dikes could be constructed and better sealed, if necessary to pond water, but this could create serious stability problems.

Vicksburg

41. A formal interview of Vicksburg District personnel was not conducted as part of this program, but informal discussions were held during a special testing visit at Yazoo City, Mississippi. This District is becoming very involved in new land disposal projects as a result of large navigation and flood control programs along the Mississippi River and its tributaries. Since confined land disposal operations are relatively new to Vicksburg District personnel, a progressive approach is followed in developing advanced disposal methods. At Yazoo City, Mississippi, high dikes are used to construct compartmentalized disposal sites (Figures A23 and A24) which cover an area of approximately 20 acres and allow deep ponding of water. The confined areas are long and narrow with a 4-to-1 or 5-to-1 length-to-width ratio; 80 percent of the surface area is in the first compartment, and the remaining 20 percent forms the final settling pond. A 100-foot-long rectangular weir with a fixed crest elevation is rigidly placed in the dike between the two compartments and is designed to maintain a two-inch head at the crest. Details on weir construction are shown in Figure A23. A plastic liner was installed on the nappe side of the weir to prevent sloughing of the cross-dike. A laterally slit discharge pipe (Figure A24) was used experimentally to allow better dispersion of the slurry influent, but this was eventually abandoned due to frequent clogging by roots and other debris.

42. The unique disposal areas described above will be discussed separately, because special dye-dispersion tests were conducted at these sites (see Part III). Of importance is the fact that the Vicksburg District has undertaken a special research program in designing and testing containment areas in order to constantly improve their performance to meet future demanding requirements. Automatic sampling equipment is being installed at all major points of the dredging and disposal operations. Data obtained from the physical-chemical analysis of the samples will be used to evaluate the parameters affecting the performance of



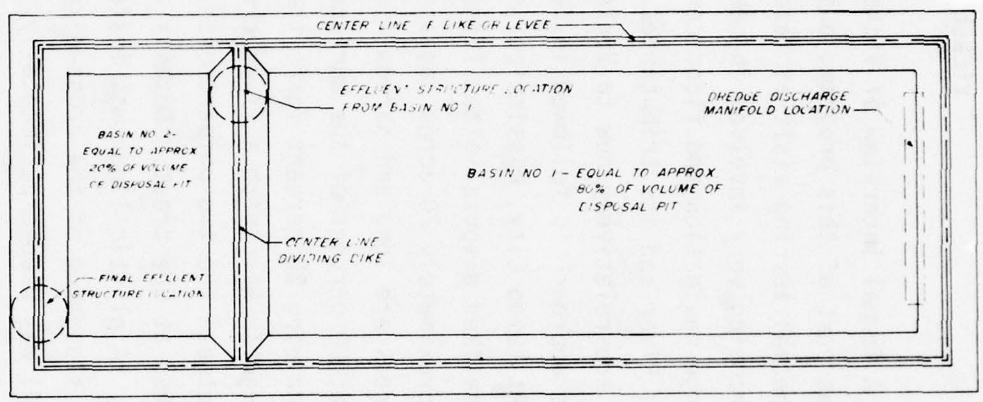
△ * NOTE:
FOR DISPOSAL AREA NO. 6 WHERE THE DIKE ELEVATION EXCEEDS 15 FT., 1 ON 4 SIDE SLOPES WILL BE REQUIRED.

TYPICAL CROSS SECTION

DREDGED MATERIAL DISPOSAL AREA

AREAS NOS. 5, 6 & 7 △

SCALE: HORZ 1 IN. = 20 FT.
VERT. 1 IN. = 10 FT.



TYPICAL PLAN

DREDGED MATERIAL DISPOSAL AREA

NOT TO SCALE

NOTE: STRUCTURE LOCATIONS ARE INTENDED TO SHOW ONLY RELATIVE PROXIMITY TO EACH OTHER

△ DIKE ELEV. TABULATION

DISPOSAL AREA NO.	DIKE ELEV. (FT.)
6	111.0
3	114.0
7	108.0
4	113.0
△ 5	112.0

BERM CONTROL TABULATION

DISPOSAL AREA NO.	DISTANCE (FT.)
1 & 2	135'
3 & 4 △	90'
△ 5, 6, 7 & 8	70'

U. S. ARMY ENGINEER DISTRICT, VICKSBURG
CORPS OF ENGINEERS
VICKSBURG, MISSISSIPPI

UPPER AUXILIARY CHANNEL ALTERNATIVE
CHANNEL IMPROVEMENT ITEM 1 - A
TYPICAL CROSS SECTIONS

Figure A23. General layout of Yazoo City disposal area

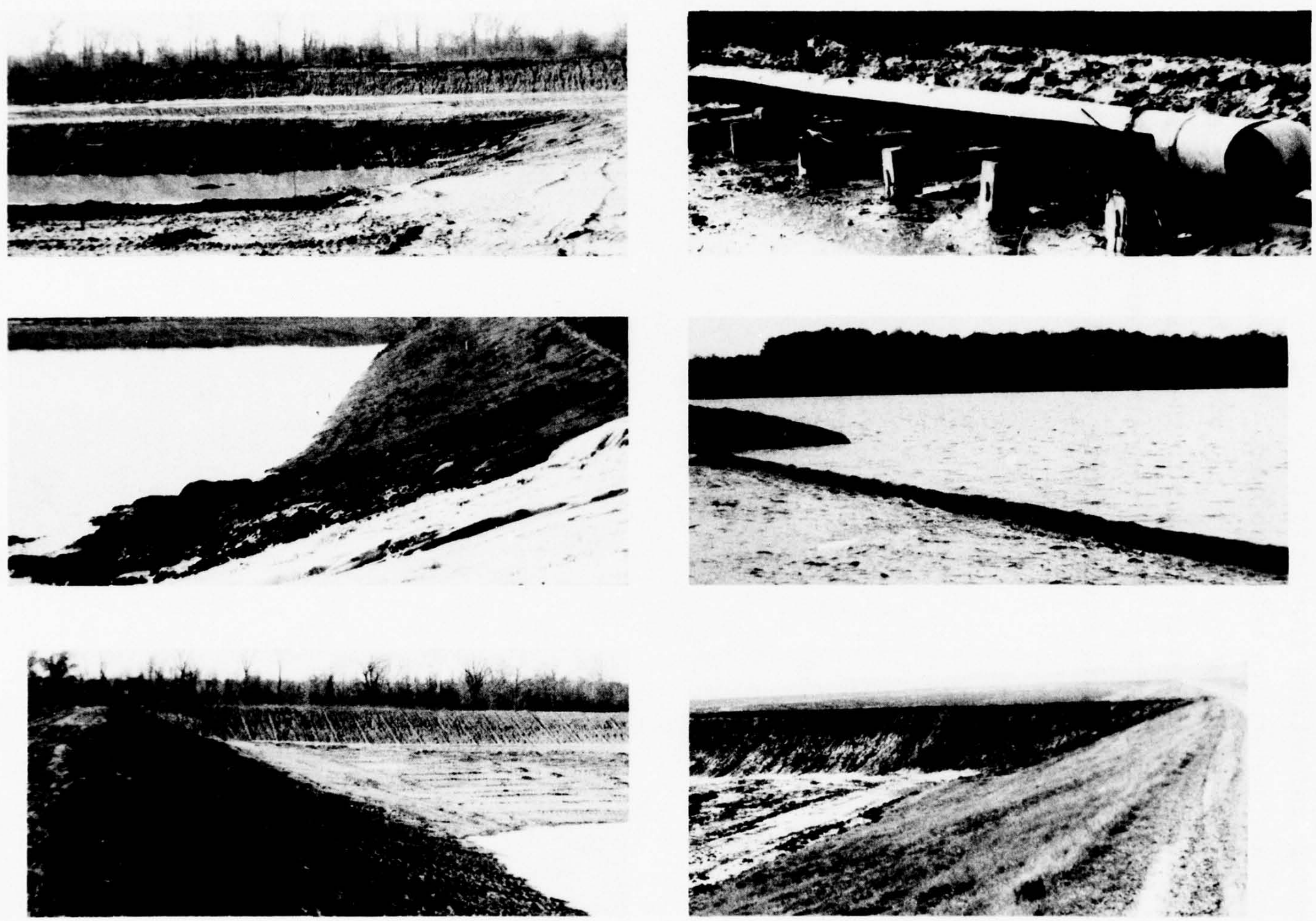


Figure A24. Photos of Yazoo City disposal area

disposal areas so that design improvements can be implemented. This program is being conducted by personnel with sanitary engineering backgrounds, and designs similar to those used for the treatment of municipal wastewaters are being employed. The cost of this program is high, but this type of effort should pay off in future savings and efficiencies. Possible modifications to the compartmentalized structure and a new advanced weir design have already emerged from this effort, and these are discussed in greater detail in other sections of this report.

APPENDIX B: MATHEMATICAL ANALYSIS OF BASIN HYDRAULICS

1. The hydraulic efficiency of a sedimentation basin is directly influenced by the flow domain prevailing in the basin. For the proper analysis and design of a basin, it is necessary to obtain a thorough understanding of the anticipated flow pattern, velocity field, and retention time. The theoretical methods advanced in the following paragraphs allow a qualitative and quantitative analysis of these factors and provide the designer with a reasonable degree of predictive capability. All mathematical formulations are adopted to describe conditions that prevail in dredged material confinement areas.

Theoretical Analysis of Flow Patterns

2. Flow in a dredged material confinement area is induced by (a) through-flow from the discharge pipe to the outlet weir and (b) wind blowing over the water surface. In the typical disposal area represented schematically in Figure B1, the average depth of water, $h\ (x, y)$, is very small with respect to the horizontal dimensions, L_1 and L_2, of the basin, and the change in the water surface elevation, $\eta\ (x, y)$, induced by wind is small with respect to the average depth of the basin. To facilitate the formulation of mathematical expressions to determine the flow patterns in a given basin, the following assumptions are introduced:

a. The effect due to the rotation of earth is neglected.

b. The fluid is homogeneous and incompressible.

c. Steady-state flow conditions prevail.

d. The convective acceleration is negligible.

e. The vertical pressure distribution is hydrostatic.

f. Horizontal turbulent stresses are neglected.

g. The eddy viscosity is constant.

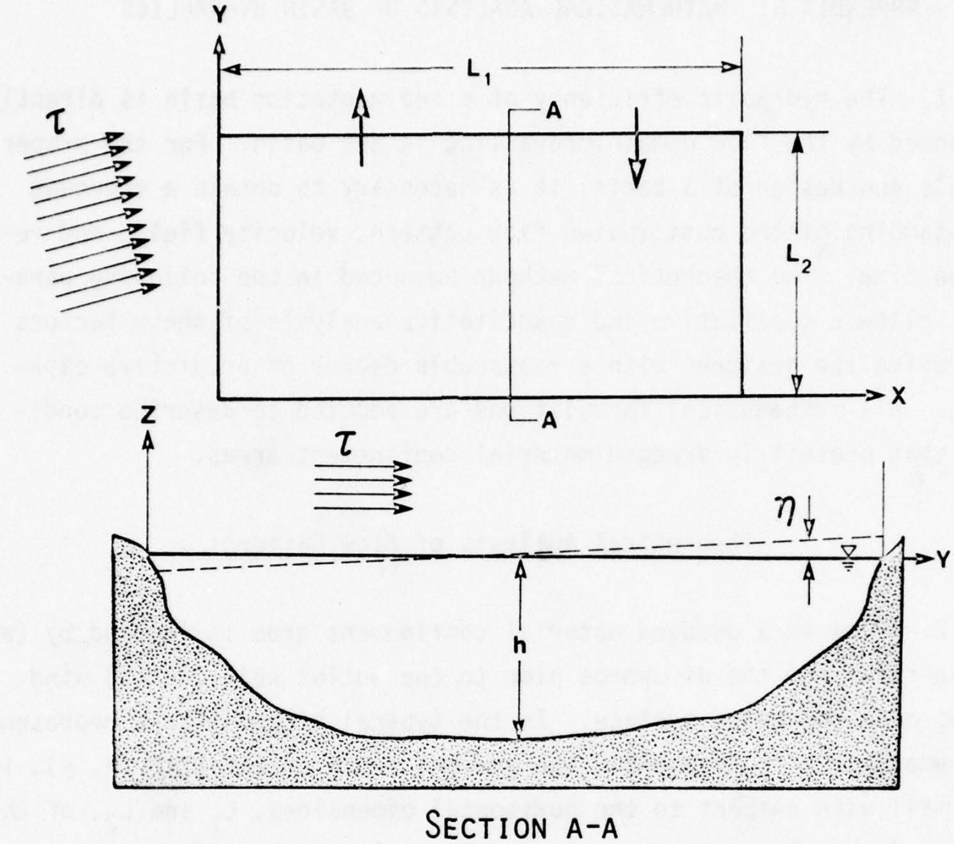


Figure B1. Schematic representation of dredged material confinement area

3. The Coriolis force is an important factor in the analysis of circulation patterns in large bodies of water, such as the Great Lakes and the oceans, but it has a negligible effect on basins of a size comparable to disposal areas. According to Liggett and Hadjitheodorou (1969) and Liu and Perez (1971), assumptions d, e, and f are valid for shallow lakes or basins. The assumption of constant eddy viscosity is not supported by substantial theoretical considerations or experimental data; however, Liggett (1970) has concluded that the variation of eddy viscosity with depth has an insignificant effect on the flow pattern and it is believed that this assumption is suitable for the purposes of this analysis.

4. According to the aforementioned assumptions, the Reynolds equations of turbulent flow (neglecting inertial forces) can be written as

$$\varepsilon \frac{\partial^2 u}{\partial z^2} - \frac{1}{\rho} \frac{\partial p}{\partial x} = 0 \tag{B1}$$

$$\varepsilon \frac{\partial^2 v}{\partial z^2} - \frac{1}{\rho} \frac{\partial p}{\partial y} = 0 \tag{B2}$$

$$-\frac{1}{\rho} \frac{\partial p}{\partial z} - g = 0 \tag{B3}$$

where u and v are the horizontal velocity components in the x and y directions, respectively; ρ is the mass density; p is the pressure; g is the acceleration of gravity; and ε is the eddy viscosity coefficient. The equation of continuity is expressed as

$$\frac{du}{dx} + \frac{dv}{dy} + \frac{dw}{dz} = 0 \tag{B4}$$

where w is the vertical component of the velocity. Equation B4 can be used to calculate the vertical velocity component, w, once the horizontal velocity components, u and v, are determined, as explained in the following paragraph.

5. The velocity distribution, u and v, can be determined by integrating Equations B1 and B2 for the following boundary conditions

$$\tau_{zx} = \rho\,\varepsilon \frac{du}{dz} \tag{B5}$$

at $z = \eta$ (surface)

$$\tau_{zy} = \rho\,\varepsilon \frac{dv}{dz} \tag{B6}$$

$$u = v = 0 \qquad \text{at } z = -h \text{ (bottom)} \tag{B7}$$

where τ_x and τ_y are the components of the wind stress in the horizontal direction, x and y, respectively. Since $\eta << h$, the integration yields

$$u = \frac{1}{2\varepsilon\rho} \frac{\partial p}{\partial x} \left(z^2 - h^2\right) + \frac{\tau_{zx}}{\varepsilon\rho} \left(z + h\right) \tag{B8}$$

$$v = \frac{1}{2\varepsilon\rho} \frac{\partial p}{\partial y} \left(z^2 - h^2\right) + \frac{\tau_{zy}}{\varepsilon\rho} \left(z + h\right) \tag{B9}$$

These equations give the horizontal velocity distribution in terms of the pressure gradients, $\partial p/\partial x$ and $\partial p/\partial y$, and the wind stress. The pressure gradients can be obtained by first introducing the horizontal transport functions, U and V, defined as

$$U = \int_{-h}^{0} u \, dz \quad \text{and} \quad V = \int_{-h}^{0} v \, dz \tag{B10}$$

Introduced next is the transport stream function Ψ, which is defined as partial derivatives:

$$\frac{\partial \Psi}{\partial y} = U \quad \text{and} \quad \frac{\partial \Psi}{\partial x} = -V \tag{B11}$$

Substitution of Equations B8 and B9 into Equations B10 and B11 yields

$$\frac{\partial p}{\partial x} = -\frac{3\varepsilon\rho}{h^3} \frac{\partial \Psi}{\partial y} + \frac{3}{2h} \tau_{zx} \tag{B12}$$

$$\frac{\partial p}{\partial y} = \frac{3\varepsilon\rho}{h^3} \frac{\partial \Psi}{\partial x} + \frac{3}{2h} \tau_{zy} \tag{B13}$$

Cross differentiation of Equations B12 and B13 with an appropriate arrangement of terms yields

$$\frac{\partial^2\Psi}{\partial x^2} + \frac{\partial^2\Psi}{\partial y^2} = \frac{3}{h}\left(\frac{\partial h}{\partial x}\frac{\partial\Psi}{\partial x} + \frac{\partial h}{\partial y}\frac{\partial\Psi}{\partial y}\right) - \frac{h}{2\varepsilon\rho}\left[h\left(\frac{\partial\tau_{zy}}{\partial x} - \frac{\partial\tau_{zx}}{\partial y}\right) - \left(\tau_{zy}\frac{\partial h}{\partial x} - \tau_{zx}\frac{\partial h}{\partial y}\right)\right] \tag{B14}$$

The solution of Equation B14 for the stream function, Ψ, requires (a) the geometry and the bottom topography, h (x, y), of the basin; (b) the surface wind stress, τ (x,y); and (c) the boundary conditions for the stream function, Ψ (x, y), prescribed for the given locations of discharge pipe and outlet weir. Once the stream function, Ψ (x, y), is obtained from Equation B14, the pressure gradients can be calculated from Equations B12 and B13, and the velocity components can be evaluated from Equations B8 and B9.

6. Equation B14 is a two-dimensional, second-order, linear partial differential equation with variable coefficients. For an arbitrary geometry and bottom topography of the basin and an arbitrary wind stress field, numerical techniques must be employed to obtain a solution. However, a number of realistic simplifications will be introduced to allow the stream function, Ψ, to be determined analytically. In the first of these simplifications, the wind stress over the basin surface will be assumed constant and Equation B14 reduces to

$$\frac{\partial^2\Psi}{\partial x^2} + \frac{\partial^2\Psi}{\partial y^2} = \frac{3}{h}\left(\frac{\partial h}{\partial x}\frac{\partial\Psi}{\partial x} + \frac{\partial h}{\partial y}\frac{\partial\Psi}{\partial y}\right) + \frac{h}{2\varepsilon\rho}\left(\tau_{zy}\frac{\partial h}{\partial x} - \tau_{zx}\frac{\partial h}{\partial y}\right) \tag{B15}$$

For a shallow disposal area, a constant depth may be assumed throughout the entire basin, and Equation B15 is further reduced to

$$\frac{\partial^2\Psi}{\partial x^2} + \frac{\partial^2\Psi}{\partial y^2} = 0 \tag{B16}$$

Equation B16 is the Laplace equation and can be solved analytically for some simple basin geometries (note that, for the case of uniform wind and constant depth, the equation satisfied by the stream function is identical to that governing two-dimensional potential flow).

General Solution for Stream Function

7. For a typical disposal area of rectangular shape, constant depth, uniform wind stress over the surface, and arbitrary location of inlets and outlets along the perimeter, the boundary conditions for the stream function, Ψ, can be defined as (see Figure B1)

$$\left.\begin{aligned} &\Psi = J_1\ (y);\ \text{at } x = 0, \\ &\Psi = J_2\ (y);\ \text{at } x = L_1, \\ &\Psi = K_1\ (x);\ \text{at } y = 0, \text{ and} \\ &\Psi = K_2\ (x);\ \text{at } y = L_2 \end{aligned}\right\} \qquad \text{(B17)}$$

where J_1, J_2, K_1, and K_2 are functions to be specified. Using the method of separation of variables, solution to Equation B16 can be written as

$$\Psi\ (x,\ y) = \sum_{n=1}^{\infty} \left\{ \frac{2 \sin \frac{n\pi x}{L_1}}{L_1 \sinh \frac{n\pi L_2}{L_1}} \int_0^{L_1} \left[K_1\ (\epsilon) \sinh \frac{n\pi\left(L_2 - y\right)}{L_1} + K_2\ (\epsilon) \sinh \frac{n\pi y}{L_1} \right] \sin \frac{n\pi\epsilon}{L_1}\ d\epsilon \right.$$

$$\left. + \frac{2 \sin \frac{n\pi y}{L_2}}{L_2 \sinh \frac{n\pi L_1}{L_2}} \int_0^{L_2} \left[J_1\ (\epsilon) \sinh \frac{n\pi\left(L_1 - x\right)}{L_2} + J_2\ (\epsilon) \sinh \frac{n\pi x}{L_2} \right] \sin \frac{n\pi\epsilon}{L_2}\ d\epsilon \right\} \qquad \text{(B18)}$$

The integrals in Equation B18 can be obtained for certain simple functions of J_1 (y), J_2 (y), K_1 (x), and K_2 (x), and the solution for the stream function, Ψ, can be expressed in terms of an infinite series which can be readily evaluated.

Solutions of the Stream Function for Some Particular Cases

8. The solutions presented below provide the theoretical foundation for the physical interpretations, discussions, and recommendations presented in Part IV. Four different basin configurations, which vary with respect to the relative location of the inlet and outlet points and the number of outlet points, were selected.

9. Case 1: One Inlet and One Outlet Along the Same Side. For this case, one inlet and one outlet point are located on the same side, as shown in Figure B2, and the boundary conditions for the stream function, Ψ, are

$$\left.\begin{aligned} & J_1(y) = J_2(y) = K_1(x) = 0 \\ & K_2(x) = \begin{cases} 0; & 0 \le x \le a_1 \\ \dfrac{Q}{b_1}(x - a); & a_1 \le x \le (a_1 + b_1) \\ Q; & (a_1 + b_1) \le x \le (L_1 - a_2 - b_2) \\ \dfrac{Q}{b_2}(L_2 - a_2 - x); & (L_1 - a_2 - b_2) \le x \le (L_1 - a_2) \\ 0; & (L_1 - a_2) \le x \le L_1 \end{cases} \end{aligned}\right\} \tag{B19}$$

where Q is the discharge rate. In Equation B19 a uniform velocity distribution is assumed across the inlet and outlet. For turbulent flow, this assumption is reasonable except near the ends of the inlet and outlet points. Substituting the boundary conditions into Equation B18 and performing the integration, we obtain the solution

$$\Psi(x, y) = \frac{2Q}{\pi^2}\left(\frac{L_1}{b_1}\right)\sum_{n=1}^{\infty}\left\{\frac{1}{n^2}\left[\sin\frac{(a_1 + b_1)n\pi}{L_1} - \sin\frac{n\pi a_1}{L_1} - \left(\frac{b_1}{b_2}\right)\sin\frac{(L_1 - a_2)n\pi}{L_1} + \left(\frac{b_1}{b_2}\right)\sin\frac{(L_1 - a_2 - b_2)n\pi}{L_1}\right]\frac{\sin\frac{n\pi x}{L_1}\sinh\frac{n\pi y}{L_2}}{\sinh\left(\frac{n\pi L_2}{L_1}\right)}\right\} \tag{B20}$$

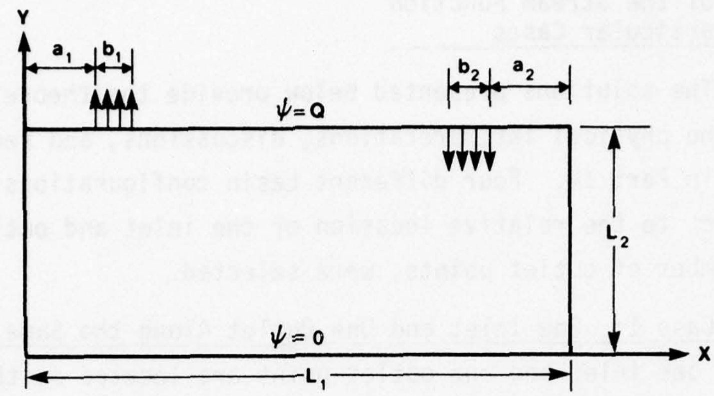


Figure B2. Basin with one inlet and one outlet on same side

10. <u>Case 2: One Inlet and One Outlet on Different Sides</u>. For this case, shown in Figure B3, the boundary conditions are

$$\left.\begin{aligned}
&K_1(x) = J_2(y) = 0 \\
&J_1(y) = \begin{cases} 0; & 0 \le y \le a_1 \\ \dfrac{Q}{b_1}(y - a_1); & a_1 \le y \le (a_1 + b_1) \\ Q; & (a_1 + b_1) \le y \le L_2 \end{cases} \\
&K_2(x) = \begin{cases} Q; & 0 \le x \le (L_1 - a_2 - b_2) \\ \dfrac{Q}{b_2}(L_1 - a_2 - x); & (L_1 - a_2 - b_2) \le x \le (L_1 - a_2) \\ 0; & (L_1 - a_2) \le x \le L_1 \end{cases}
\end{aligned}\right\} \quad \text{(B21)}$$

and the stream function, Ψ, is

$$\Psi = (x, y) = 2Q\sum_{n=1}^{\infty}\left\{\frac{\sin\frac{n\pi x}{L_1}\sinh\frac{n\pi y}{L_1}}{n\pi\sinh\left(\frac{n\pi L_2}{L_1}\right)}\left[1 - \frac{1}{n\pi}\left(\frac{L_1}{b_2}\right)\sin\frac{n\pi\left(L_1 - a_2\right)}{L_1} + \frac{L_1}{n\pi b_2}\sin\frac{n\pi\left(L_1 - a_2 - b_2\right)}{L_1}\right.\right.$$

$$\left.\left. + \frac{\sin\frac{n\pi y}{L_2}\sinh\frac{n\pi\left(L_1 - x\right)}{L_2}}{n\pi\sinh\left(\frac{n\pi L_1}{L_2}\right)}\left[\frac{L_2}{n\pi b_2}\sin\frac{n\pi\left(a_1 + b_1\right)}{L_2} - \frac{L_2}{n\pi b_1}\sin\frac{n\pi a_1}{L_2} - \cos n\pi\right]\right\} \quad \text{(B22)}$$

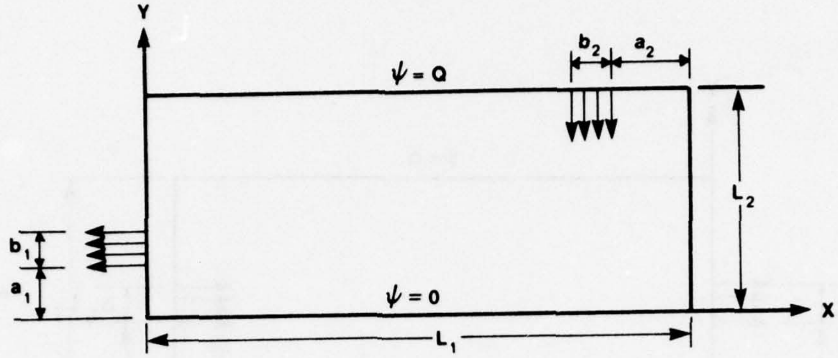


Figure B3. Basin with one inlet and one outlet on different sides

11. Case 3: One Inlet and One Outlet on Opposite Sides. For this case, shown in Figure B4, the boundary conditions are

$$
\left.
\begin{aligned}
&K_1\ (x) = 0\\
&K_2\ (x) = Q\\
&J_1\ (y) = \begin{cases} 0; & 0 \le y \le a_2\\ Q\ \left(\dfrac{y - a_2}{b_2}\right); & a_2 \le y \le (a_2 + b_2)\\ 0; & (a_2 + b_2) \le y \le L_2 \end{cases}\\
&J_2\ (y) = \begin{cases} 0; & 0 \le y \le a_1\\ Q\ \left(\dfrac{y - a_1}{b_2}\right); & a_1 \le y \le (a_1 + b_1)\\ Q; & (a_1 + b_1) \le y \le L_2 \end{cases}
\end{aligned}
\right\} \qquad \text{(B23)}
$$

and the stream function, ψ, is

$$
\begin{aligned}
\psi\ (x,\ y) = Q \sum_{n=1}^{\infty} \Bigg\{ & \frac{2\left(1 - \cos n\pi\right) \sin \frac{n\pi x}{L_1} \sinh \frac{n\pi y}{L_1}}{n\pi \sinh\left(\frac{n\pi L_2}{L_1}\right)} + \frac{2 \sin \frac{n\pi y}{L_2}}{n\pi \sinh\left(\frac{n\pi L_1}{L_2}\right)}\\
& \bullet \Bigg[\sinh \frac{n\pi\left(L_1 - x\right)}{L_2} \left[\frac{L_2}{b_2\, n\pi}\left(\sin \frac{n\pi\left(a_2 + b_2\right)}{L_2} - \sin \frac{n\pi\, a_2}{L_2}\right) - \cos n\pi\right]\\
& + \sinh \frac{n\pi x}{L_2} \left[\frac{L_2}{b_1\, n\pi}\left(\sin \frac{n\pi\left(a_1 + b_1\right)}{L_2} - \sin \frac{n\pi\, a_1}{L_2}\right) - \cos n\pi\right]\Bigg]\Bigg\}
\end{aligned}
\qquad \text{(B24)}
$$

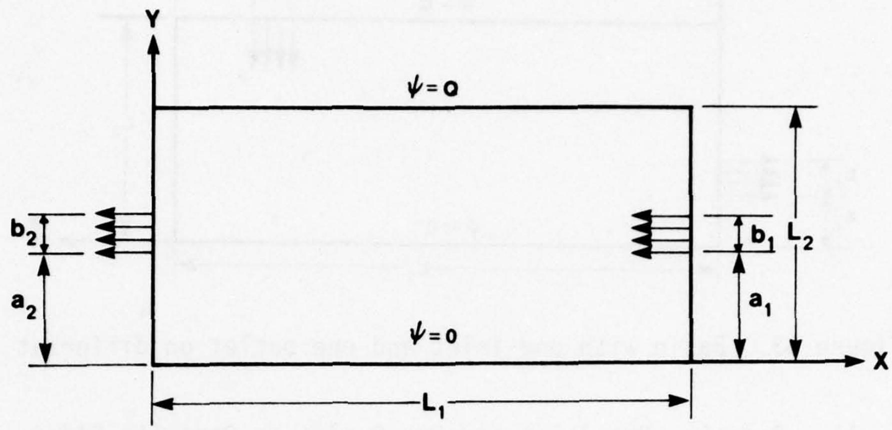


Figure B4. Basin with one inlet and one outlet on opposite sides

12. Case 4: One Inlet and Two Outlets on Opposite Sides. As shown in Figure B5, the boundary conditions for this case are

$$\left.\begin{array}{l}
K_1\ (x) = 0 \\
K_2\ (x) = 0 \\
J_1\ (y) = \begin{cases} 0; & 0 \le y \le a_2 \\ \dfrac{Q}{2b_2}\ (y - a_2); & a_2 \le y \le (a_2 + b_2) \\ \dfrac{Q}{b}; & (a_2 + b_2) \le y \le (L_2 - a_2 - b_2) \\ \dfrac{Q}{2b_2}\ \ y - \left[(L_2 - a_2 - 2b_2)\right]\ ; & (L_2 - a_2 - b_2) \le y \le L_2 - a_2 \\ Q; & (L_2 - a_2) \le y \le L_2 \end{cases} \\
J_2\ (y) = \begin{cases} 0; & 0 \le y \le a_1 \\ \dfrac{Q}{b_1}\ (y - a_1); & a_1 \le y \le (a_1 + b_1) \\ Q; & (a_1 + b_1) \le y \le L_2 \end{cases}
\end{array}\right\} \qquad \text{(B25)}$$

Assuming that the two outlets have equal strength and are located symmetrically about the center line of the basin, the stream function, Ψ, can be expressed as

$$\psi\,(x,\,y) = Q \sum_{n=1}^{\infty} \left\{ \frac{2(1 - \cos n\pi)\,\sin \frac{n\pi x}{L_1} \sinh \frac{n\pi y}{L_1}}{n\pi \sinh \left(\frac{n\pi L_2}{L_1}\right)} + \frac{2 \sin \frac{n\pi y}{L_2} \sinh \frac{n\pi x}{L_2}}{n\pi \sinh \left(\frac{n\pi L_1}{L_2}\right)} \right.$$

$$\cdot \left[\frac{L_2}{n\pi b_1} \left(\sin \frac{n\pi\left(a_1 + b_1\right)}{L_2} - \sin \frac{n\pi a_1}{L_2} \right) - \cos n\pi \right] + \frac{2 \sin \frac{n\pi y}{L_2} \sinh \frac{n\pi\left(L_1 - x\right)}{L_2}}{n\pi \sinh \left(\frac{n\pi L_1}{L_2}\right)}$$

$$\left. \cdot \left[\frac{L_2}{2b_2 n\pi} \left(\sin \frac{n\pi\left(a_2 + b_2\right)}{L_2} - \sin \frac{n\pi a_2}{L_2} + \sin \frac{\left(L_2 - a_2\right)n\pi}{L_2} - \sin \frac{\left(L_2 - a_2 - b_2\right)n\pi}{L_2} \right) - \cos n\pi \right] \right\} \qquad \text{(B26)}$$

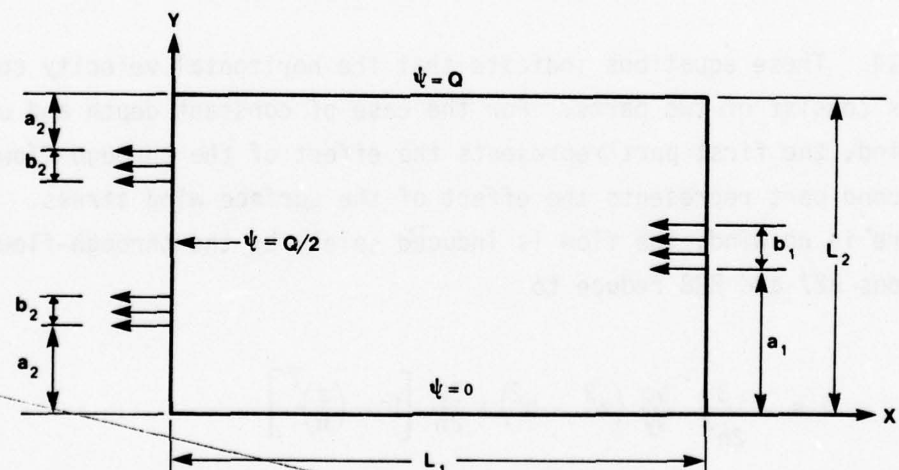


Figure B5. Basin with one inlet and two outlets on opposite sides

Velocity Field and Wind Effects

13. The three-dimensional velocity field in a shallow basin can be expressed in terms of the transport stream function, ψ. For a basin with arbitrary geometry, bottom topography, through-flow from the inlet to the outlet weir, and wind stress, Equation B14 can be solved for Ψ by numerical techniques. For the case of a steady uniform wind and a constant basin depth, Equation B14 reduces to Laplace's equation (Equation B16), which can be solved analytically for some basin configurations, as was shown in the preceding paragraphs. Once the solution for the transport stream function is known, the horizontal velocity components, u and v, can be evaluated by substituting Equations B12 and B13 into Equations B8 and B9, respectively. Thus,

$$u = -\frac{3}{2h^3}\frac{\partial\psi}{\partial y}\left(z^2 - h^2\right) + \frac{\tau_{zx}}{\varepsilon\rho}\left[\frac{3}{4h}\left(z^2 - h^2\right) + \left(z + h\right)\right] \qquad \text{(B27)}$$

and

$$v = \frac{3}{2h^3}\frac{\partial\psi}{\partial x}\left(z^2 - h^2\right) + \frac{\tau_{zy}}{\varepsilon\rho}\left[\frac{3}{4h}\left(z^2 - h^2\right) + \left(z + h\right)\right] \qquad \text{(B28)}$$

14. These equations indicate that the horizontal velocity components consist of two parts. For the case of constant depth and uniform wind, the first part represents the effect of the through-flow and the second part represents the effect of the surface wind stress. Thus, if there is no wind, the flow is induced solely by the through-flow and Equations B27 and B28 reduce to

$$u = -\frac{3}{2h^3}\frac{\partial\psi}{\partial y}\left(z^2 - h^2\right) = \frac{3U}{2h}\left[1 - \left(\frac{z}{h}\right)^2\right] \qquad \text{(B29)}$$

and

$$v = \frac{3}{2h^3}\frac{\partial\psi}{\partial x}\left(z^2 - h^2\right) = \frac{3V}{2h}\left[1 - \left(\frac{z}{h}\right)^2\right] \qquad \text{(B30)}$$

The surface velocity components are

$$u_o = \frac{3U}{2h} = \frac{3}{2}\ \bar{u} \tag{B31}$$

$$v_o = \frac{3V}{2h} = \frac{3}{2}\ \bar{v} \tag{B32}$$

where $\bar{u}$ and $\bar{v}$ are the x and y components, respectively, of the average velocity over a vertical section. For convenience, Equations B29 and B30 can be written in dimensionless form as

$$\frac{u}{u_o} = \frac{v}{v_o} = 1 - \left(\frac{z}{h}\right)^2 \ ; \qquad -1 \le \frac{z}{h} \le o \tag{B33}$$

Shown in Figure B6 is the vertical distribution of the horizontal velocity components due to through-flow.

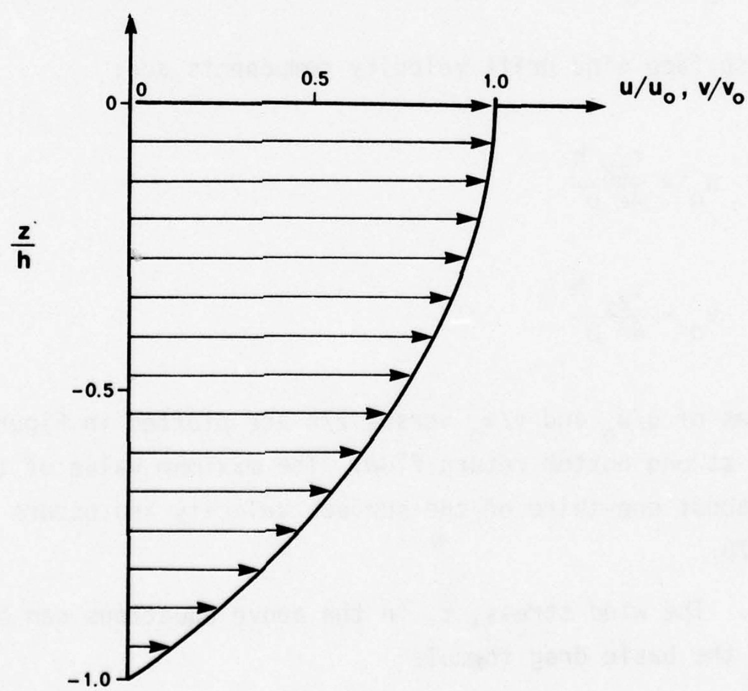


Figure B6. Distribution of horizontal velocity due to through-flow

15. The velocity induced by a uniform wind in a constant-depth basin is expressed as:

$$u = \frac{\tau_{zx}}{\varepsilon\rho}\left[\frac{3}{4h}\left(z^2 - h^2\right) + \left(z + h\right)\right] \tag{B34}$$

and

$$v = \frac{\tau_{zy}}{\varepsilon\rho}\left[\frac{3}{4h}\left(z^2 - h^2\right) + \left(z + h\right)\right] \tag{B35}$$

When circulation in a confined basin is generated by wind, the bottom return flow compensates for the forward flow in the upper layers, and the depth-integrated horizontal transports, U and V, are equal to zero. In dimensionless form, Equations B34 and B35 are expressed as

$$\frac{u}{u_o} = \frac{v}{v_o} = 1 + 4\left(\frac{z}{h}\right) + 3\left(\frac{z}{h}\right)^2 ; \quad -1 \leq \frac{z}{h} \leq o, \tag{B36}$$

and the surface wind drift velocity components are:

$$u_o = \frac{\tau_{zx}\, h}{4\varepsilon\ \rho} \tag{B37}$$

and

$$v_o = \frac{\tau_{zy}\, h}{4\varepsilon\ \rho} \tag{B38}$$

The values of u/u_o and v/v_o versus z/h are plotted in Figure B7 and indicate a strong bottom return flow. The maximum value of the return flow is about one-third of the surface velocity and occurs at a depth of about 0.7h.

16. The wind stress, τ, in the above equations can be estimated by using the basic drag formula

$$\tau = c\ \rho_a\ V_w^2 \tag{B39}$$

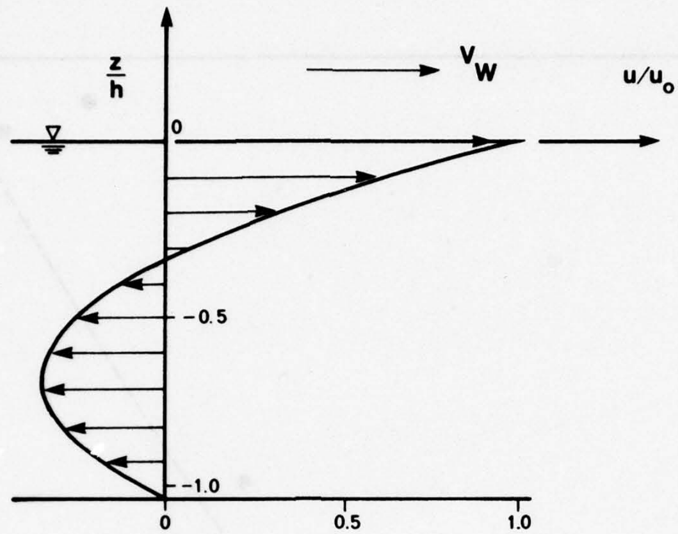


Figure B7. Vertical distribution of wind drift velocity

in which ρ_a is the mass density of air (approximately equal to 1.25 x 10^{-3} g/cm^3); V_w is the wind speed measured at an elevation of 10 meters above the water surface; and c is the drag coefficient which varies with wind speed. According to Priestley (1959), a c value of 1.2 x 10^{-3} appears to be a good approximation for a 10- to 25-mph wind. Very limited information is available with respect to values for the coefficient of eddy viscosity, but Liu and Perez (1971) indicated that this coefficient has values on the order of 0.5 cm^2/sec for basins that are 1 to 5 feet deep. There is evidence to suggest that the eddy viscosity coefficient increases substantially with depth (Karaushev, 1960), and the accepted values for the Great Lakes range from 20 to 80 cm^2/sec. Values of the eddy viscosity coefficient, as reported by Karaushev (1960), are plotted in Figure B8.

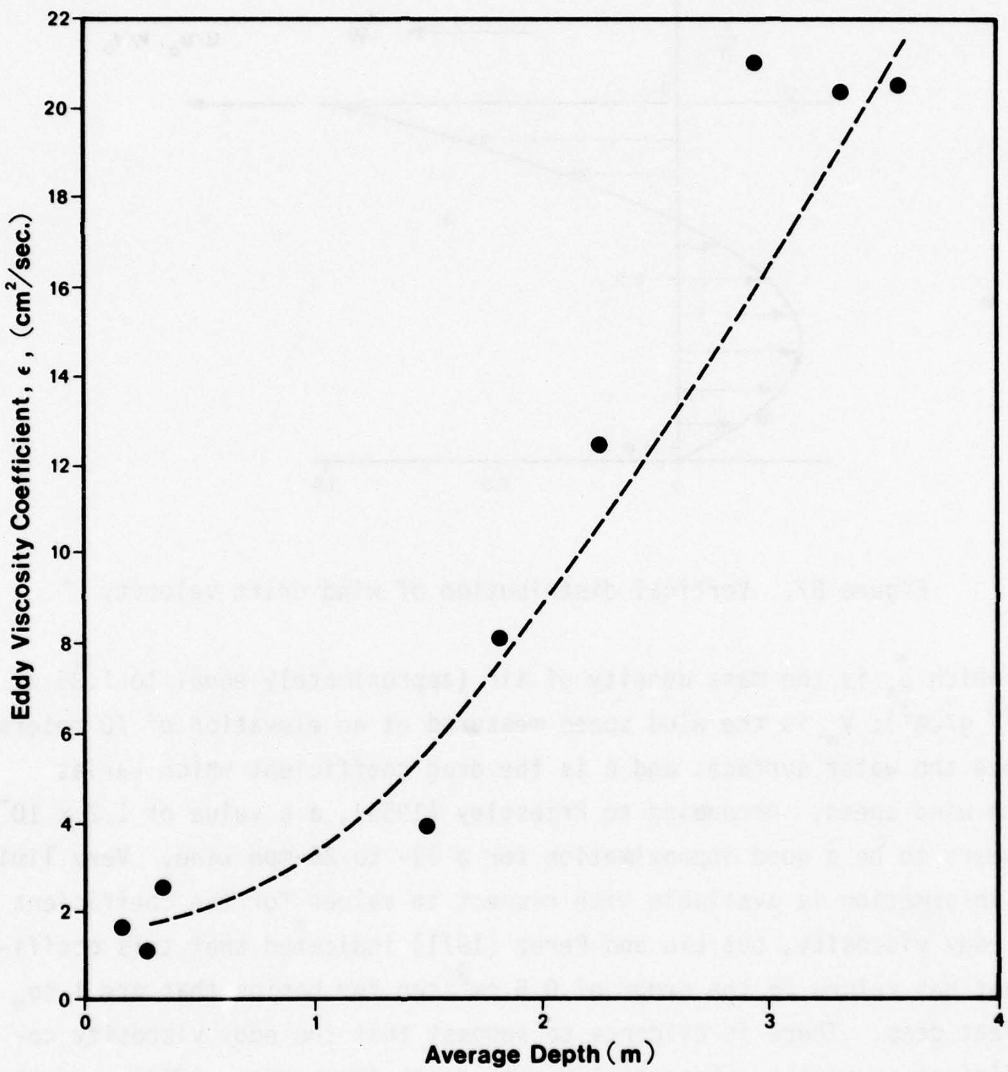


Figure B8. Experimental values for coefficient of eddy viscosity in terms of average depth (After Karaushev, 1960)

Retention Time

17. Retention time is defined as the time interval during which a particle travels from the inlet to the outlet location of the basin. As shown in Figure B9, the retention time, T_r , can be expressed as the sum of the elementary times, dt, required by the particle to travel a distance, dL, at a speed of $|V|$. Along a pathline,

$$\frac{dL}{|V|} = \frac{dx}{u} = \frac{dy}{v} \tag{B40}$$

which yields

$$dt = \frac{dx}{u} = \frac{dy}{v} \tag{B41}$$

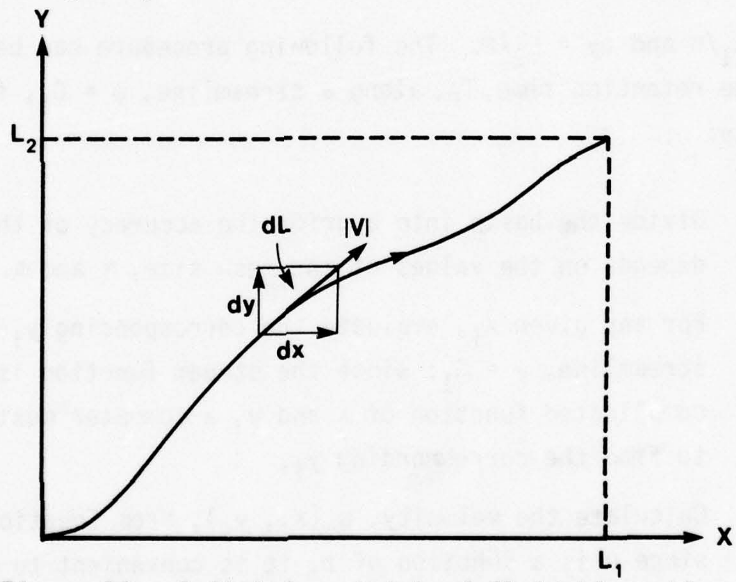


Figure B9. Definition sketch of fluid kinematics

The retention time, T_r, is determined by integrating Equation B41 along a pathline to obtain

$$T_r = \int_0^{T_r} dt = \int_0^{L_1} \frac{dx}{u} = \int_0^{L_2} \frac{dy}{v} \tag{B42}$$

For steady flow the pathlines coincide with the streamlines and, by using the solutions for the stream function and the velocity, the retention time can be calculated from Equation B42. Due to the complexity of the velocity field, this integral cannot be evaluated analytically, and numerical techniques must be employed. Accordingly, Equation B42 is approximated along a streamline by

$$T_r = \sum_{i=1}^{n} \frac{(\Delta x)_i}{u_i} \tag{B43}$$

and

$$T_r = \sum_{j=1}^{m} \frac{(\Delta y)_j}{v_j} \tag{B44}$$

where $\Delta x = L_1/n$ and $\Delta y = L_2/m$. The following procedure can be used to calculate the retention time, T_r, along a streamline, $\psi = C_1$, for zero wind velocity:

a. Divide the basin into a grid; the accuracy of the result depends on the values of the mesh size, n and m.

b. For any given x_i, evaluate the corresponding y_i on the streamline, $\psi = C_1$; since the stream function is a highly complicated function of x and y, a computer must be used to find the corresponding y_i.

c. Calculate the velocity, $u\ (x_i,\ y_i)$, from Equation B29; since u is a function of z, it is convenient to use the surface retention time.

d. Using Equation B43, calculate the retention time along $\psi = C_1$.

18. The distribution of the retention times in a basin can be calculated by dividing the basin into k parts, using predetermined streamlines, ψ_j, so that the flow rate will be equal to Q/k between

any two adjacent streamlines, ψ_j and ψ_{j+1} $\left[j = 0, 1, 2, \ldots, (k - 1)\right]$. Once the retention time is calculated along each streamline, the average surface retention time between adjacent streamlines can be obtained. Since the retention time is inversely proportional to the velocity, its average value at any depth z, between two adjacent streamlines, ψ_j and ψ_{j+1}, can be calculated by using Equation B33. For computational purposes, the vertical section can be divided into Y parts, each part representing an equal discharge rate; thus, between any two adjacent streamlines and depths z_i and z_{i+1} $\left[i = 0, 1, 2, \ldots, (Y - 1)\right]$, the discharge rate is $Q/(kY)$. The average retention time for this part of the basin (i. e., between depths z_i and z_{i+1} and streamlines ψ_j and ψ_{j+1}) should be calculated at the depth where the flow rate equals $(2i + 1)\ Q/(2kY)$ and along the streamline $\psi = (2j + 1)\ Q/2k$. Thus, there are $(k \times Y)$ numbers of average retention times calculated according to the above procedure. These $(k \times Y)$ numbers are then ranked according to their magnitudes; the shortest time is ranked first. Finally, a dye-release time span is selected as a moving interval to construct the distribution of retention time in the basin. The dye-release time span can be interpreted as the length of the time of injecting dye in a dispersion experiment.

19. A modified procedure must be used when there are streamlines that give double values of y_i for each corresponding x_i, such as the streamlines $\psi/Q = 0.1, 0.2$, etc., in Figure 12 in Part IV. For this case, the basin has to be divided into separate parts. For those parts of the basin where there is only a one-to-one correspondence between x and y, Equation B43 should be used for calculation. In the rest of the basin one should use Equation B44 to calculate the retention time. The total retention time is the sum of the retention times in each part of the basin.

APPENDIX C: WEIR AND FLOW CHARACTERISTICS LITERATURE REVIEW

1. This appendix presents a review of relevant literature on important factors affecting the hydraulic characteristics of sedimentation basins, including weir operation, basin shapes, and selective withdrawal of effluents. The first part of this literature survey is restricted to a hydraulic analysis of weirs as flow-control structures in relation to sedimentation basins. Weirs used as outlet structures in sedimentation basins are divided into three major categories: (a) sharp- or broad-crested rectangular weirs, (b) polygonal weirs, and (c) shaft-type weirs.

Sharp-Crested Rectangular Weirs

2. A common feature of sharp-crested weirs used in practice is that the nappe of water separates from the crest (i. e. the overflow is free). In the hydraulic analysis of weir performance, the most important objective is the determination of flow discharge over the weir in terms of the controlling factors. The most widely used relationship is based on the application of Bernoulli's law and assumes nonsubmerged conditions, as shown in Figure C1. After accounting for local head losses, this relationship takes the form

$$Q = \frac{2}{3} \sqrt{2g} \; C_c \, L \left[\left(H + \frac{V_a^2}{2g} \right)^{3/2} - \left(\frac{V_a^2}{2g} \right)^{3/2} \right] \qquad \text{(C1)}$$

where Q is the flow rate; L is the length of the weir; H is the head over the crest; g is the acceleration of gravity; V_a is the approach velocity; and C_c is the coefficient of contraction.

3. The effect of both the approach velocity, V_a, and the contraction, C_c, may be represented by a single coefficient, C_D, such that

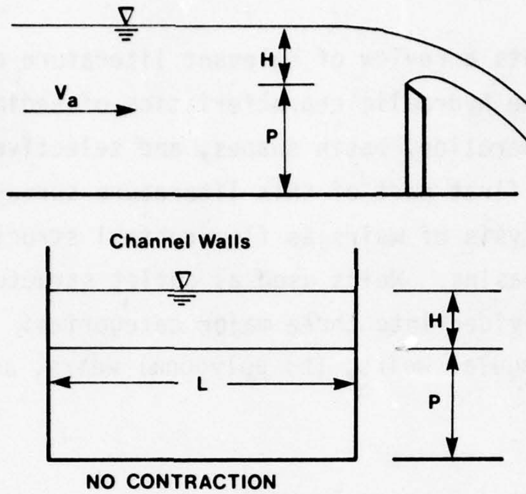


Figure C1. Sharp-crested rectangular weirs

$$Q = \frac{2}{3} \sqrt{2g}\, C_D\, L\, H^{3/2} \tag{C2}$$

which, by incorporating the constant $\frac{2}{3}\sqrt{2g}$, becomes the coefficient C_D'

$$Q = C_D'\, L\, H^{3/2} \tag{C3}$$

where $C_D' = \frac{2}{3} \sqrt{2g}\, C_D$ is the overflow coefficient. The measurement of head, H, must be made a sufficient distance upstream from the crest to be beyond the zone of appreciable drawdown. As a rule of thumb (Rouse, 1949, p. 213) this distance should be at least 3H. If it is more practical to measure the head above the crest, adjustment has to be made for the effect of drawdown. According to the classical work by Rehbock (1929), the relationship between the depth of water over the crest, h, and the head, H, is

$$H = \frac{h}{0.85} \tag{C4}$$

4. The overflow and discharge coefficients, C_D' and C_D, are dependent on (a) the relative dimensions (geometry) of the weir, (b) the height of the weir crest above the bottom, (c) the side contraction, (d) the approach velocity, (e) the direction of approach, and (f) the head over the weir crest. One of the best known formulas for calculating C_D' was advanced by Rehbock (1929):

$$C_D' = \left(3.24 + 0.43\,\frac{H}{P} + \frac{0.018}{H}\right) \qquad \text{(C5)}$$

where H is the head over the crest and P is the height of the weir (see Figure C1). More recently an extensive study was undertaken by Kindsvater and Carter (1959) to learn more about the effect of various factors on the overflow coefficient. As a result of this study, the investigators recommended new relationships for the calculation of these factors. Several international standards are based on their observations, and therefore a review of their results is of merit.

5. By introducing the concepts of effective weir length, L_e, and effective head, H_e, Kindsvater and Carter (1959) proposed the following general formula to calculate the flow rate, Q:

$$Q = C_D'\,L_e\,H_e^{1.5} \qquad \text{(C6)}$$

where $H_e = H + 0.003$ ft and $L_e = L + k_B$. The values of k_B are shown in Figure C2, and the values of C_D' are plotted on Figure C3. If the weir is not contracted, we have

$$C_D' = 3.22 + 0.40\,\frac{H}{P} \qquad \text{(C7)}$$

which is the so-called alternate Rehbock formula. It is evident that for small values of H/P, the various formulae show very small disagreement, but for H/P > 1 the deviations become increasingly more signifi-

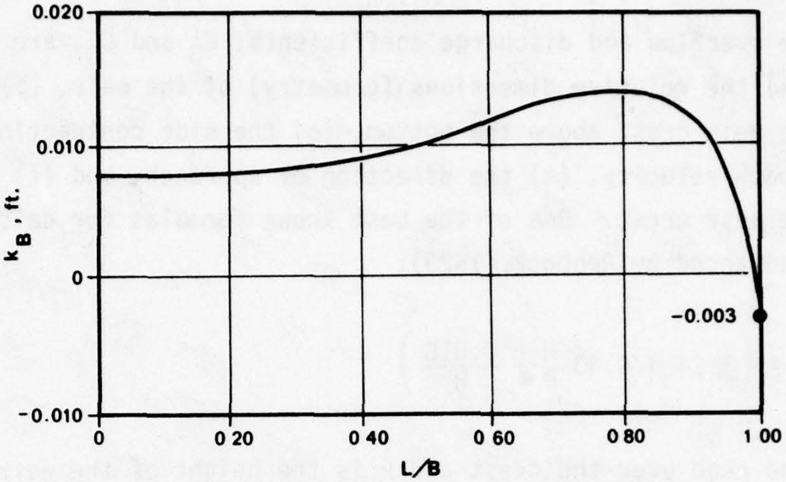


Figure C2. Values of k_B versus contraction ratio

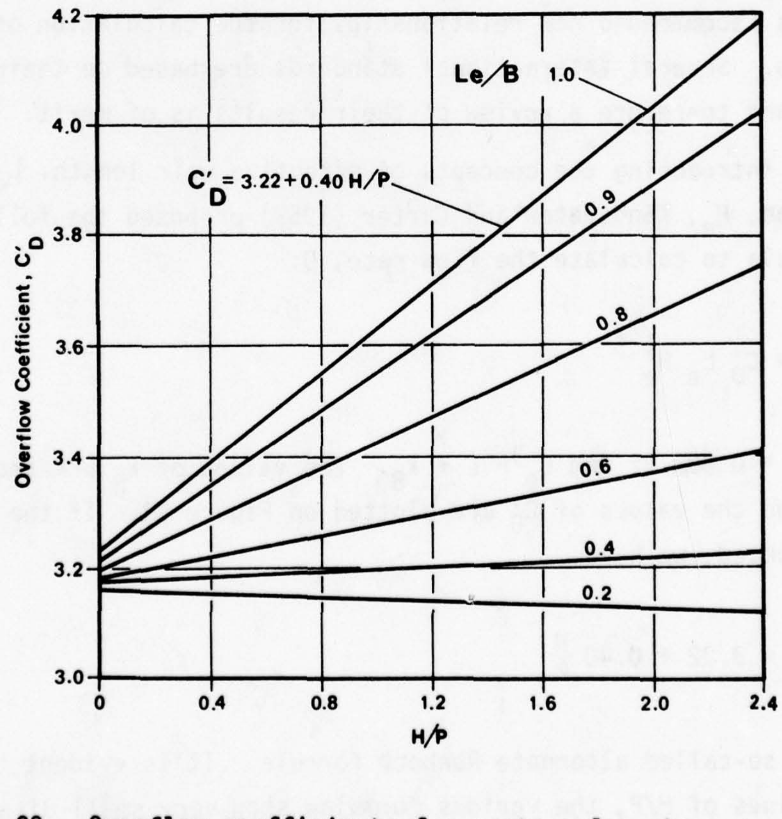


Figure C3. Overflow coefficients for rectangular sharp-crested weirs

cant. The degree of contraction is expressed by the ratio L/B, as shown in Figure C3, where B is the width of the approaching flow. Extensive studies (Rouse, 1949, p. 215) have shown that the effect of side contraction on the discharge coefficient is insignificant if the head, H, does not exceed one-third of the crest length, L. In general, dredged material disposal basins do not operate with large heads, H, relative to either weir height, P, or crest length, L; therefore the considerations discussed above would not apply in these cases.

6. The discharge coefficients of sharp-crested weirs are sensitive to the angle that the weir makes with the vertical. In Figure C4 the relationship between the angle α and the corresponding correction factor κ (Starasolszky, 1970) is shown. If the weir tilts toward the downstream side, the effect of the angle is favorable if $\alpha < 70^0$; beyond this value the correction factor decreases rapidly. The direction of the approach velocity does not have a significant effect on the flow quantity over a weir. According to Kiselev (1950) the flow rate, Q, can be obtained by multiplying the values computed according to Equation C6 by a coefficient K, which is a function of the angle β that the approach velocity makes with the crest of the weir. Values for this coefficient are given in Table C1.

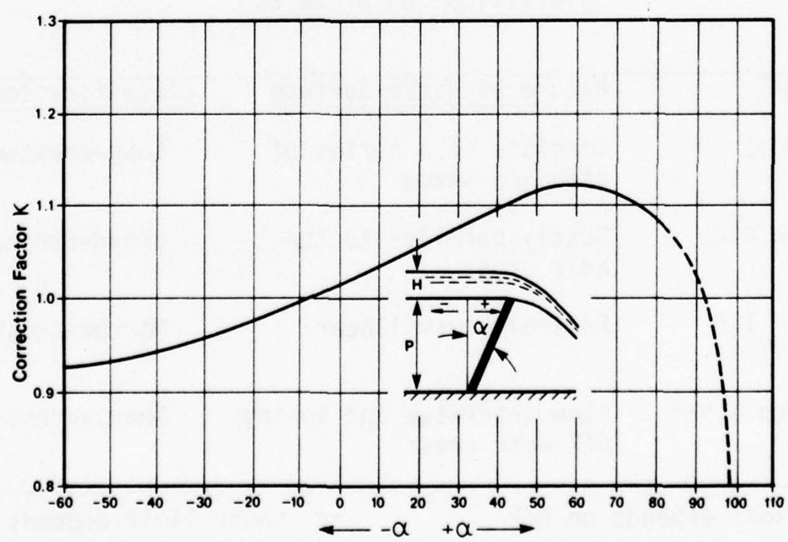


Figure C4. Effect of weir angle α

Table C1

Weir Flow Rate Correction for Nonperpendicular Flow Velocities

Angle of Approach Velocity β	15^{o}	30^{o}	45^{o}	60^{o}	90^{o}
Correction Coefficient K	0.86	0.91	0.94	0.96	1.00

Broad-Crested Rectangular Weirs

7. If the cross section of the weir is not a thin plate, but instead has a thickness T, of various shapes, the water flow (nappe) will not separate from the crest, but will adhere to it. It is customary to classify rectangular weirs in terms of the H/T ratio; accordingly, they can be subdivided into narrow-crested weirs, broad-crested weirs, and long-crested weirs (ISO/TC, 1964, 1968; Muralidhar, 1965; Rao and Shukla, 1971). The classification proposed by Rao and Shukla (1971) is shown in Table C2, and illustrated in Figure C5.

Table C2

Classification of Weirs

Value of H/T	Nature of Water Surface	Classification of Weir
$0 < H/T \le 0.1$	Consists of a series of standing waves	Long-crested weir
$0.1 < H/T < 0.4$	Mostly parallel to the weir crest	Broad-crested weir
$0.4 < H/T \le 1.5$ to 1.9*	Entirely curvilinear	Narrow-crested weir
$H/T \ge 1.5$ to 1.9**	Flow separates and springs off weir crest	Sharp-crested weir

* Upper limit depends on H/P ** Lower limit depends on H/P

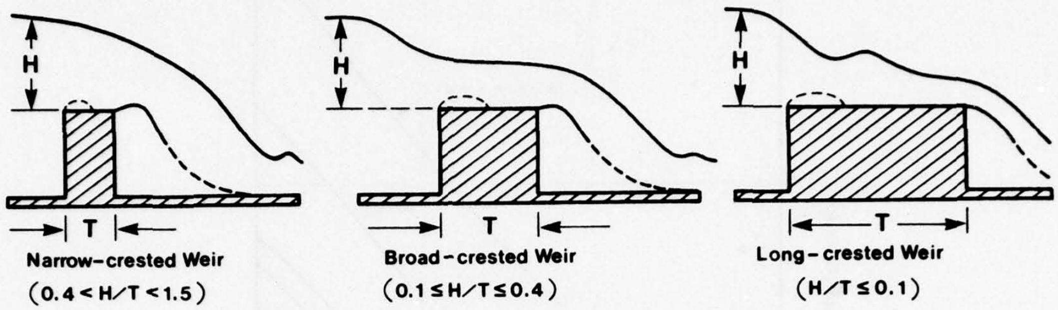


Figure C5. Flow over weirs of finite crest thickness

8. The hydraulic characteristics of weirs with a wide crest thickness have been studied by a large number of investigators. Analyses of the available literature were presented by Engle and Stainsby (1958), Muralidhar (1965), Tracy (1957) and Starasolszky (1970). The basic broad-crested weir equation is (National Bureau of Standards Special Publication 421, 1975)

$$Q = C_D \, L \, H_1^{3/2} \tag{C8}$$

or

$$Q = C_D' \, L \, H^{3/2} \tag{C9}$$

in which L is the weir length and H_1 is the total head (measured head, H, plus the effective approach velocity head). When using Equations C8 and C9, caution must be exercised in selecting values for the discharge or overflow coefficients, C_D or C_D'. For broad-crested weirs ($0.1 < H/T < 0.4$) C_D' is a constant equal to 2.62 ± 0.08, provided that the values of H/P are between 0.22 and 0.56 (British Standards Institution, 1965). For $H/T > 0.4$, the recommended correction factors, G, by which weir coefficients should be multiplied are given in Figure C6. If the weir is

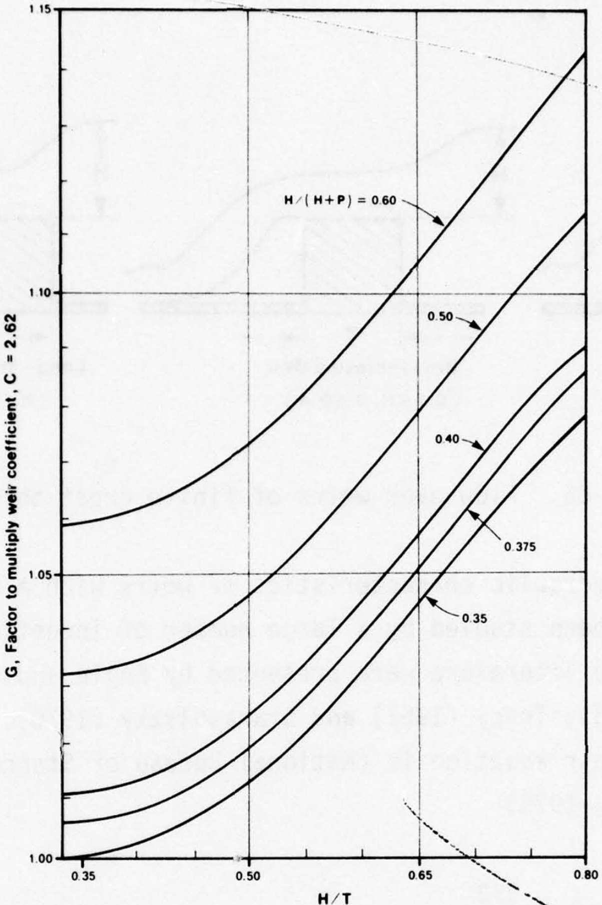


Figure C6. Discharge coefficients for square-edge, broad-crested weirs

contracted, the discharge values calculated from the above equations must also be multiplied by another factor, K_0, (Starasolszky, 1970):

$$K_0 = 1 - \frac{c'}{\left(0.2 + \frac{P}{H}\right)^{1/3}} \left(\frac{L}{B}\right)^{1/4} \left(1 - \frac{L}{B}\right) \qquad \text{(C10)}$$

in which L is the weir length; B is the width of the approach channel; P is the height of the weir; and c' is a coefficient which depends on the shape of the weir (for rectangular broad-crested weirs, c' = 0.19; for round-edged rectangular weirs, c' = 0.10).

Polygonal Weirs

9. Weirs of polygonal shape include square intake towers, labyrinth weirs, duck-bill over-falls, etc; and they are characterized by a broken axis in plan (see Figure C7). The purpose of polygonal weirs is to increase the active weir length (length of crest), thus making it possible to increase the discharge per unit length of structure for a given head, as indicated in Figure C7. Such an arrangement is advantageous if the available width and the head over the weir are limited. A number of polygonal weirs have been constructed and are in operation as outlet structures for containment areas.

10. Several authors (Aichel, 1907; Boileau, 1854; Darvas, 1971; Hay and Taylor, 1970; Indlekofer and Rouve, 1975; Kozák and Sváb, 1961; Escande and Sabathé, 1937; Gentilini, 1941) have studied the capacity of various polygonal overflow structures. The most comprehensive analysis was performed by Hay and Taylor (1970), and recently Indlekofer and Rouve (1975) studied the effect of corners on the discharge capacity of weirs.

11. Hay and Taylor (1969) developed a computer program for the analysis of labyrinth weirs and substantiated the validity of this program with laboratory tests. The performance of labyrinth weirs was evaluated by direct comparison of labyrinth weir flows, Q_L, with the corresponding sharp-crested linear weir flows, Q_N. This method of analysis is dependent on an accurate knowledge of Q_N. Hay and Taylor (1969) used the formula proposed by Kindsvater and Carter (1959) (see Equation C6):

$$Q_N = C_D' L_e H_e^{3/2} \tag{C6a}$$

They used a C_D' value given by $3.22 + 0.4 \frac{H}{P}$, where H and P are the measured head and weir crest height corresponding to the labyrinth weir discharge, Q_L. The results of their comparison are shown in Figure C8,

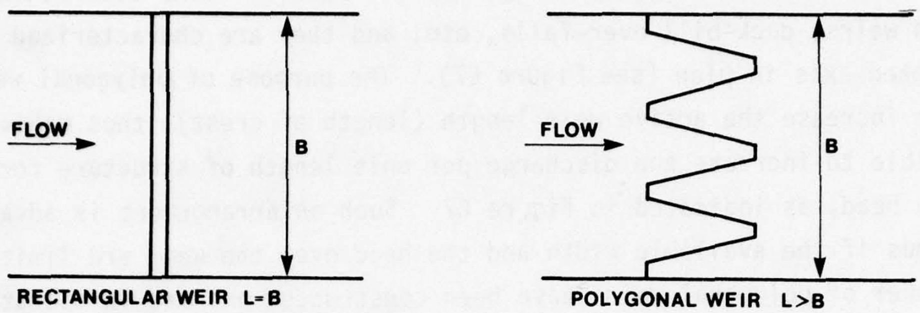


Figure C7. Effective increase of crest length in polygonal weirs

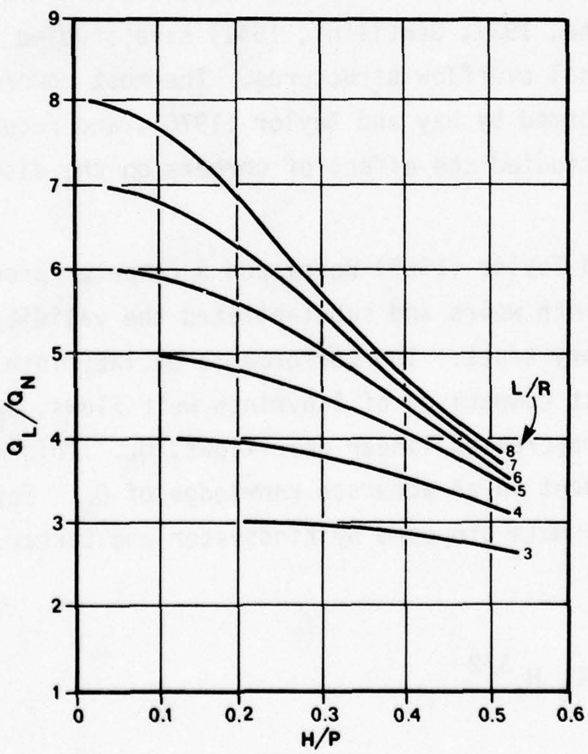


Figure C8. Relative flow rate of polygonal weirs for rectangular plan geometry

which can be used to determine the ratio Q_L/Q_N in terms of $\frac{H}{P}$ and $\frac{L}{R}$ for a weir with rectangular plan geometry, where R is the width of the corresponding rectangular sharp-crested weir and L is the length of the polygonal weir crest. This chart has been developed for $\frac{L}{R}$ ratios from 3 to 8, and it clearly indicates that, for small $\frac{H}{P}$ ratios, which is frequently the case for dredged material disposal basins, the increase of weir length by applying a polygonal arrangement will result in an almost proportional increase in the flow rate, as compared to a regular sharp-crested weir. For instance, for a crest-length magnification of 8, the value of Q_L/Q_N decreases from its initial value of 8 to 4 as H/P increases from 0 to 0.5.

Shaft-Type Weirs

12. In shaft-type weirs, the water flows over a circular or rectangular crest and discharges down a shaft (see Figure C9). Calculation of the discharge capacity of a shaft spillway is based on the same principles used for sharp-crested rectangular weirs. The flow rate may be calculated from Equation C6, where $L_e = 2\pi r$ for a circular shaft (r is the radius) and $L_e = 2a + 2b$ for a rectangular shaft (a and b are side lengths of the rectangular cross section.) It may be assumed that C_D' is equal to the values proposed for sharp-crested weirs. The chart in Figure C10 can be used to determine the values of C_D' for circular shafts and for three conditions of approach depth (Davis and Sorensen, 1969, pp. 20-32).

Weir Loading

13. In practice, maximum weir loadings are specified by standards; for example, the "Recommended Standards for Sewage Works" published by the Great Lakes-Upper Mississippi River Board of State Sanitary Engineers (1971) stipulates that "weir loadings should not exceed 10,000 gallons per day per linear foot (124 m^3/day/m) for plants

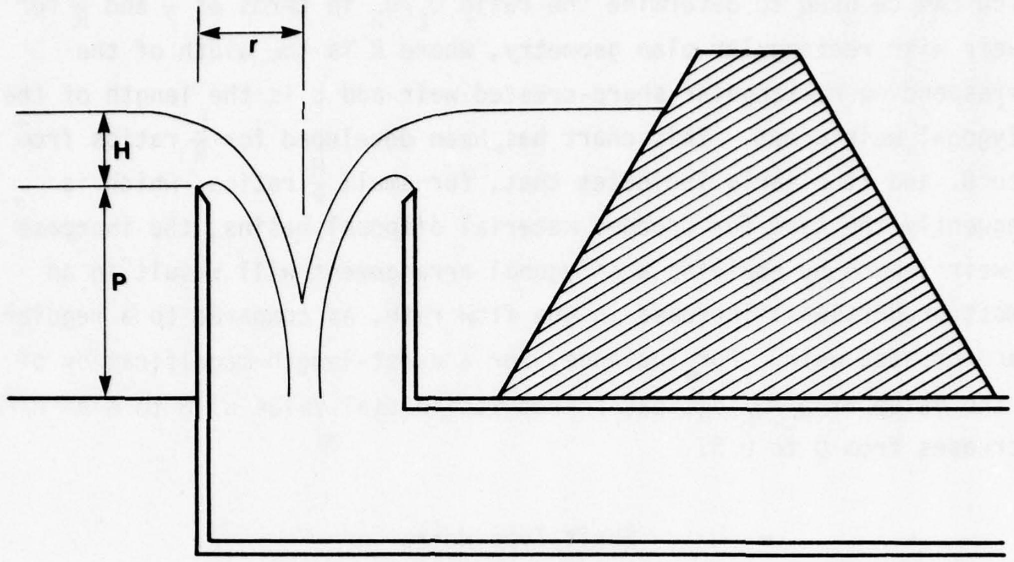


Figure C9. Shaft-type weirs

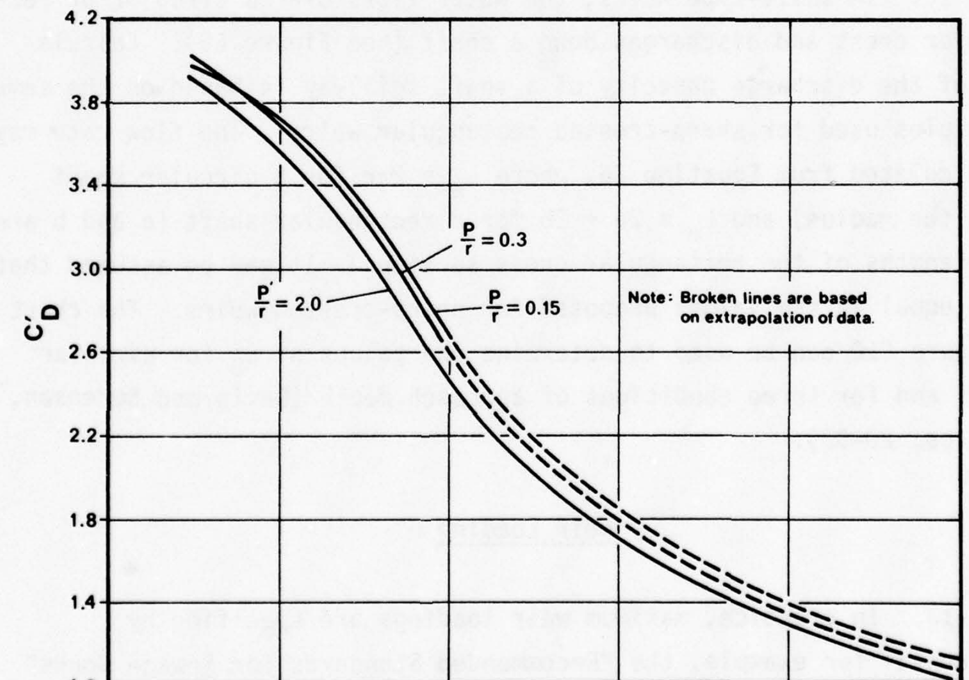


Figure C10. Values of C_D' for circular shaft weirs (After Davis and Sorensen, 1969)

designed for average flows of 1.0 mgd (3,785 m^3/day) or less. Special consideration will be given to weir loadings for flows in excess of 1.0 mgd (3,785 m^3/day), but such loadings should preferably not exceed 15,000 gallons per day per linear foot (186 m^3/day/m)." For large plants these restrictions become impractical, and a compromise must be reached; however, in many cases this compromise still results in loadings 10 times higher than the limit suggested by regulatory agencies. It may be of interest to note that the 15,000 gallons per day per linear foot limit requires that the head over the weir should not exceed one-half inch.

14. Concern for weir loading rates was first expressed by Giles (1943). Later, Anderson (1945) analyzed the results of a series of tests conducted at the Chicago Southwest Plant and established the importance of weir design for secondary settling tanks. He demonstrated that improper weir location and higher weir loading resulted in higher suspended solid content in the effluent, and this was attributed to the existence of density currents. During the same time, Gould (1945) came to a similar conclusion. One year later, the Publication titled "Sewage Treatment at Military Installations" (National Research Council, 1946) presented a theory which concluded that weir loadings have a significant effect on settling effectiveness in primary, as well as secondary, settling basins. Ingersoll et al. (1956) criticized the National Research Council theory on the basis of a comprehensive study of the performance of rectangular settling tanks. They pointed out that, unless there is a significant density current, outlet weirs can only influence settling efficiency by distorting flow patterns. According to their experiments, this influence is insignificant in primary settling tanks. Consequently, weir loading alone does not have any measurable effect on the settling efficiency of primary sedimentation tanks.

15. The conclusions of Ingersoll et al. (1956) were supported by tests conducted by Theroux and Betz (1959) on the performance of primary settling tanks. They ran two parallel tests with weir loadings of

40,000 gpd/ft and 70,000 gpd/ft, and the results indicated no measurable difference in the effectiveness of the two settling tanks. They also cited performance data on other plants and stated that "there appears to be no obvious advantage as far as suspended solids removal is concerned in using the longer weirs." In the same year (1959), the Manual of Practice on Sewage Treatment Plant Design adopted by the Water Pollution Control Federation criticized the "Ten State Standards" requirements by stating that:

a. "In primary sedimentation tanks, there is no positive evidence that weir rate per se has any significant effect on removals," and

b. "Although some authorities recommend maximum weir loadings, except on final tanks for activated sludge, there is little evidence of any limitation."

Effects of Weir Location, Wind, and Baffling

16. Less-than-full-width weirs produce contraction of flow approaching the weir and give rise to the development of eddy zones which might significantly reduce the effective surface area and the corresponding settling efficiency. Marske and Boyle (1973) conducted an extensive study of the hydraulic efficiency of chlorine contact chambers, but their objective was not to determine settling effectiveness. The types of contact basins studied were (a) rectangular secondary clarifiers, (b) circular secondary clarifiers, (c) long, narrow, and shallow channels, (d) rectangular contact basins, (e) cross-baffled serpentine basins, (f) longitudinal-baffled serpentine basins, and (g) annular ring around secondary clarifiers. These investigators studied the influence of various factors on the hydraulic performance of contact chambers, including the effects of basin depth, outlet weir configuration, serpentine baffling configuration, and length-to-width ratio. Briefly summarized, their findings were:

a. The outlet weir configuration appears to have a significant effect on the hydraulic efficiency of a contact basin. At a plant having a rectangular basin, two different types of outlet weirs (a Cipoletti weir and a sharp-crested weir) were tested. The width of the Cipoletti weir was originally only 3.5 feet; the opening was later blanked off by plywood thus creating a sharp-crested weir of 18-foot length with a significantly reduced weir loading. The flow curves shown in Figure C11 indicate that lower weir loading reduced short-circuiting and increased the available volume of the basin. The experiments imply that a sudden contraction of flow at the outlet weir would considerably reduce the settling efficiency of the tank, or, in other words, the increased weir loading has an adverse effect if the weir is not a full-length weir.

b. Wind appears to have a serious short-circuiting effect in contact basins that are long and shallow. The flow curves shown in Figure C12 are considerably different; their shapes depend on the direction of the wind, but the other hydraulic characteristics remained unchanged.

c. The effect of baffling depends on the direction of the baffles. In the past, cross baffling has been used to improve chamber effectiveness, whereas longitudinal baffling reduces the dead space areas and flow shadow areas. Consequently, longitudinally baffled basins exhibited a much lower dispersion index than cross-baffled basins, as shown in Figure C13.

d. The experiments indicated that, to achieve a practically perfect plug flow, a basin with a high length-to-width ratio (40 to 1 or higher) should be constructed. However, this ratio can be achieved by longitudinal baffling.

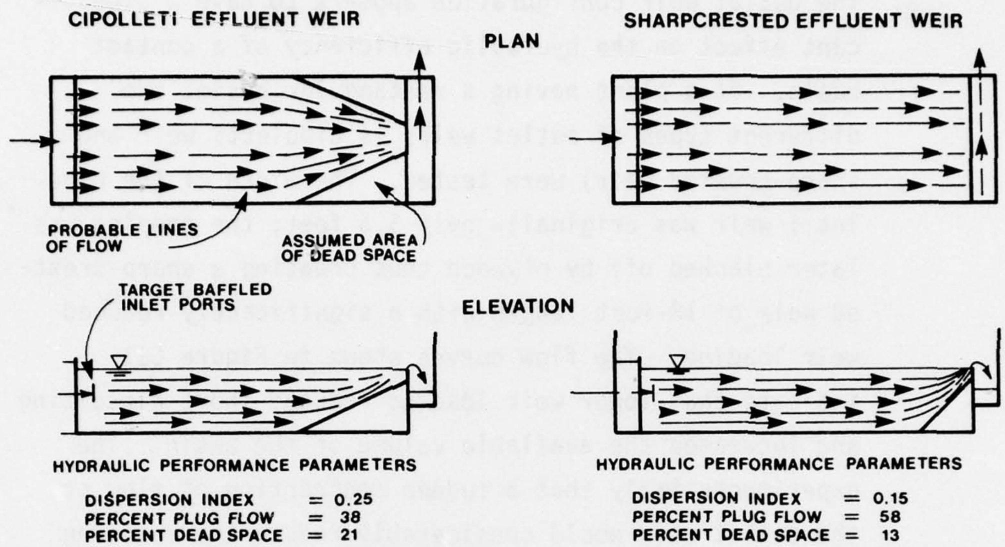


Figure C11. Influence of weir length on hydraulic efficiency (After Marske and Boyle, 1973)

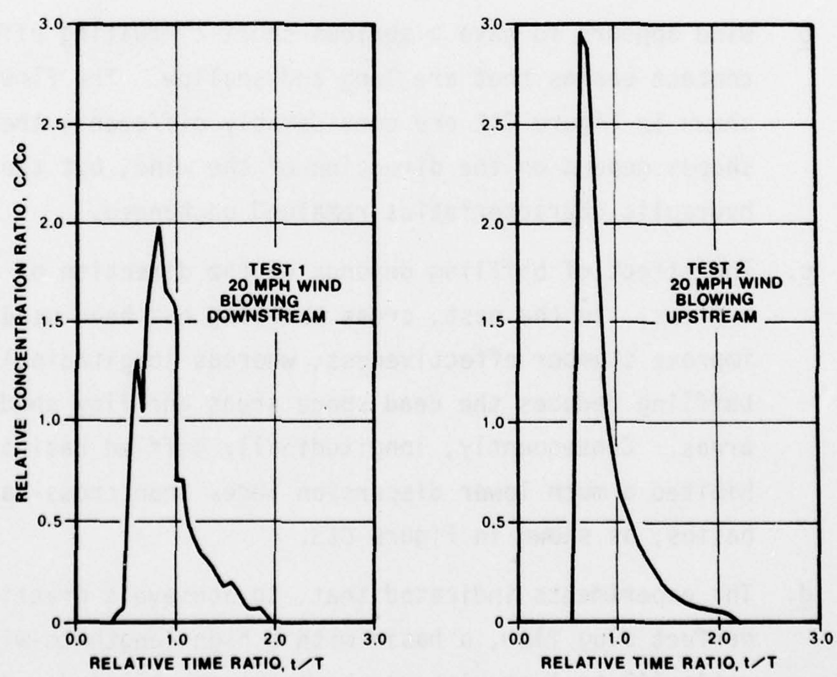


Figure C12. Influence of wind on dispersion curves for a long rectangular basin (After Marske and Boyle, 1973)

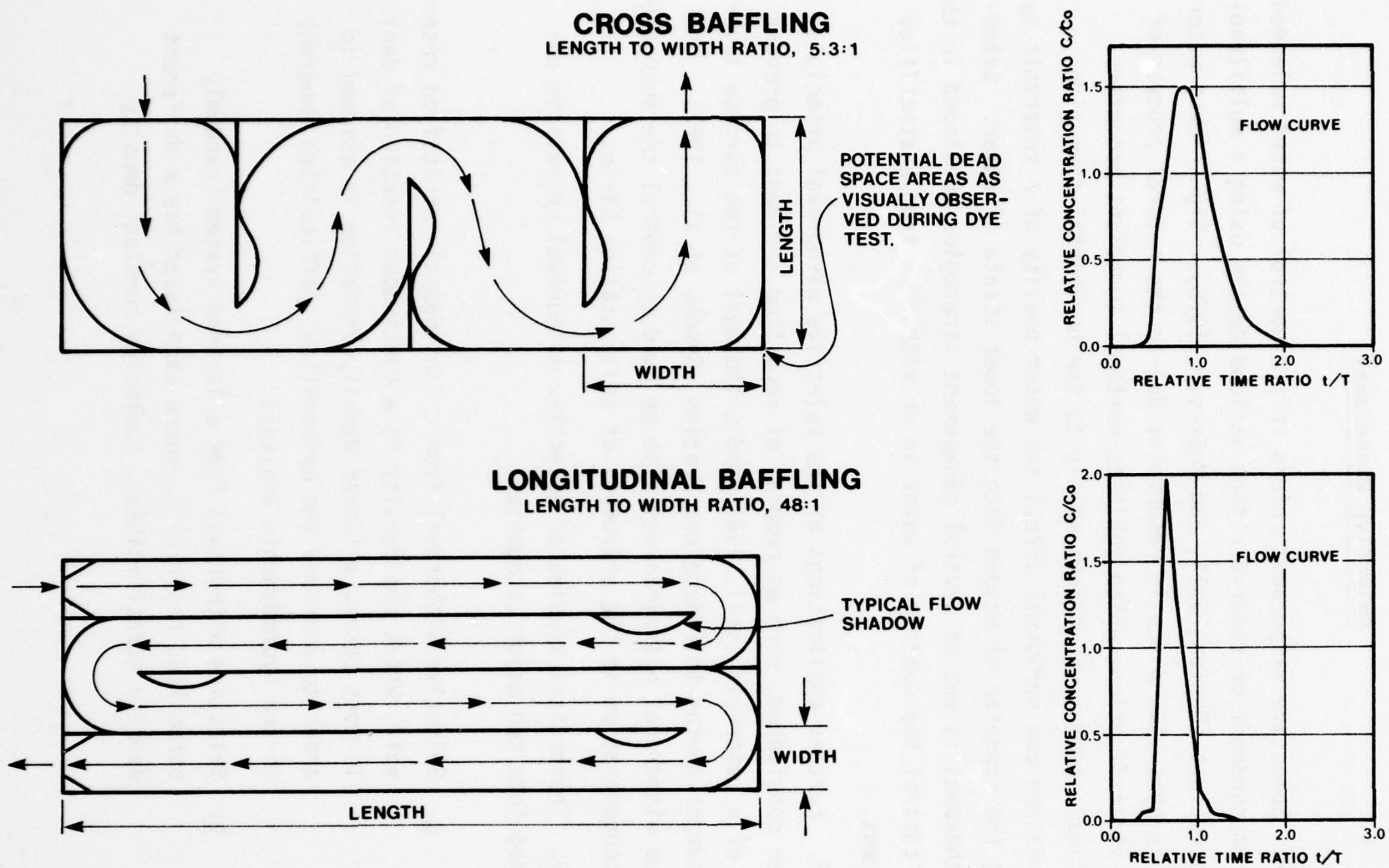


Figure C13. Effect of cross baffling versus longitudinal baffling of serpentine flow basins (After Marske and Boyle, 1973)

Selective Withdrawal

17. Selective withdrawal refers to the control of water released from an impoundment or reservoir from various depths using a multilevel outlet structure (Benton, Wall, and McKeever, 1970). Significant water quality changes take place in reservoirs due to the natural process of density stratification, which inhibits vertical turbulent processes and confines convective motions mainly to the horizontal direction. This condition can seriously affect the water quality of a reservoir by reducing the transfer of oxygen into the lower strata of water. Selective withdrawal is one of several management strategies developed in the past to control the quality of water in or withdrawn from a stratified impoundment.

18. Further applications of the selective withdrawal principle have been considered, such as removal of deposited sediment to prevent silting of a reservoir (Bell, 1942) and withdrawal of the surface layer during summer months to reduce evaporation (Symons et al., 1967). The selective withdrawal principle may also be used to control the suspended solids concentration in the effluents of sedimentation basins.

19. Theoretical studies of selective withdrawal operations can be divided into two major categories:

a. Selective withdrawal from a continuously stratified reservoir, where the density is a continuous function of depth; in most cases, a linear density variation is assumed in order to overcome the mathematical difficulties inherent in the hydrodynamic analysis.

b. Selective withdrawal from a layered system (randomly stratified reservoir), where each layer has a different density stratification, including constant density.

20. In recent years, several analytical and experimental studies have been carried out to obtain a better understanding of the hydraulics involved in selective withdrawal. Elder and Wunderlich (1969) pointed out that temperature differences are the most important factors affecting density stratification in reservoirs, although the presence of salt and turbidity may also have some effect. For conditions of complete mixing, there is very little or no density variation in the reservoir. In such a case, withdrawal through an outlet involves flow of water from the full depth of the reservoir (Figure C14a). When density stratification exists, the withdrawal current will be confined to a relatively narrow layer (Figure C14b). Imberger and Fischer (1970) and Koh (1966a) divided the withdrawal zone into four different flow regions. As shown in Figure C15, no selective withdrawal occurs in region I. Viscosity and buoyancy forces are dominant in region II, while inertia and gravity forces influence the flow conditions adjacent to the sink (region IV). In region III the flow is an intermediate case between inviscid and viscid conditions.

21. Selective withdrawal will occur in the inviscid region when the densimetric Froude number exceeds a certain critical value given by the relationship

$$F = \sqrt{\frac{U'}{\frac{g}{\rho_o} \frac{d\rho}{d\psi} d^2}} \qquad \text{(C11)}$$

where U' is the velocity far upstream of the sink; ρ_o is the fluid density at the reservoir bottom; ψ is the stream function; ρ is the density of the fluid; d is the depth of fluid in the reservoir; and g is the gravitational acceleration. Critical values for the densimetric Froude number have been determined by various investigators and are in good agreement (0.32, 0.28, and 0.29 as determined by Yih (1965), Kao (1965), and Debler and Stotta et al. (1969), respectively).

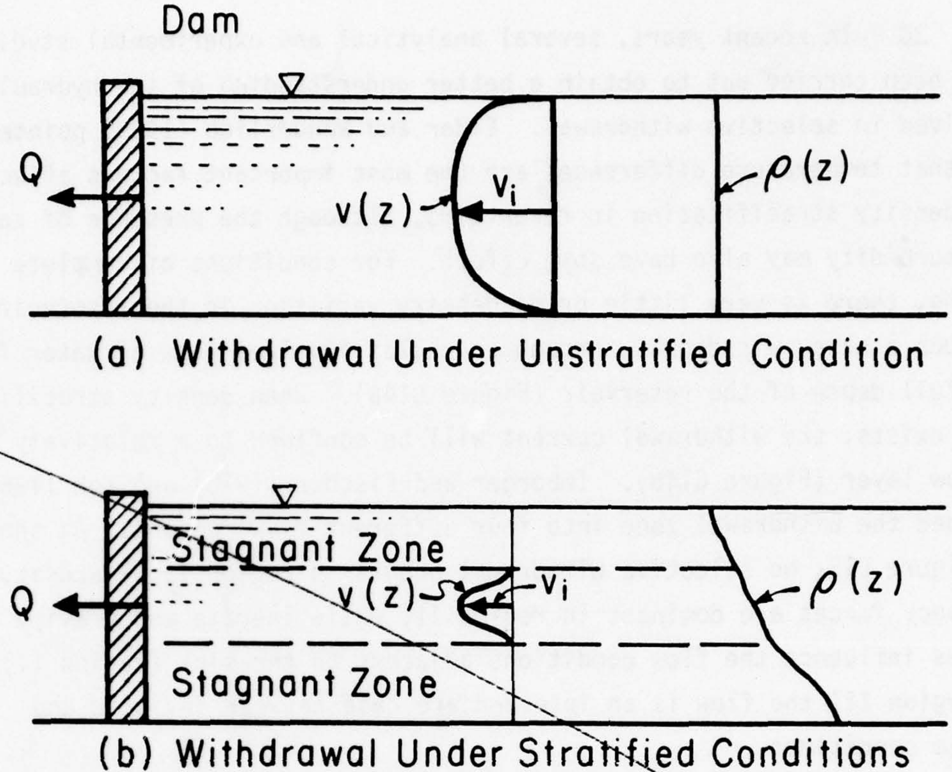


Figure C14. Mechanisms of selective withdrawal
(After Elder and Wunderlich, 1969)

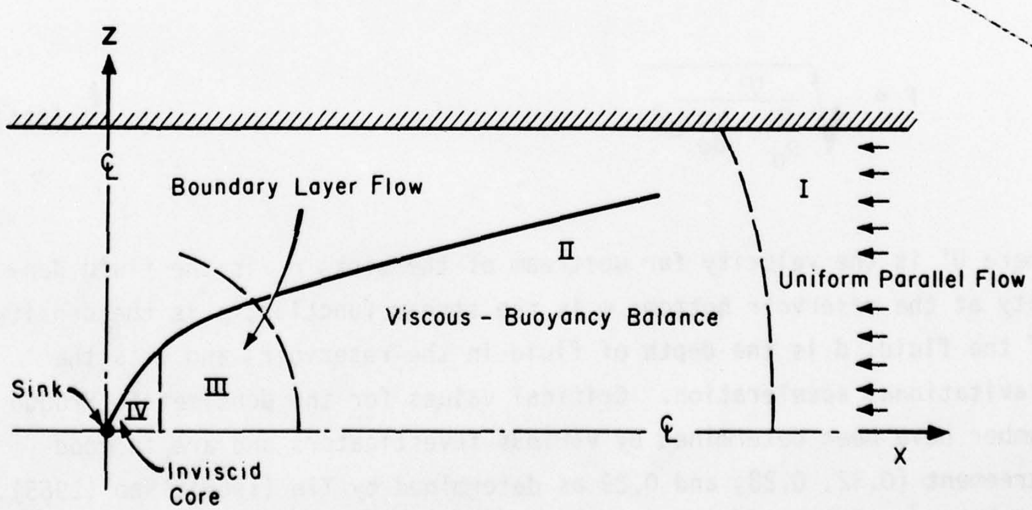


Figure C15. Structure of withdrawal layer
(After Imberger and Fischer, 1970)

22. A mathematical model employing the simplifying assumption of nondiffusive fluid was developed by Gelhar and Mascolo (1966), and their results were in good agreement with those of Koh (1966b). Imberger and Fischer (1970) incorporated the inertial effects in a mathematical model which yielded predictions that agreed closely with experimental results.

23. The planar bottom withdrawal aspects were studied by Debler (1959), Yih (1965), and Walesh and Monkmeyer (1973), each of whom used a different approach. Walesh and Monkmeyer (1973) included the effects of viscosity, but neglected the effect of mass and heat diffusion in their mathematical model, which was analyzed by the perturbation method. Laboratory experiments by Walesh (1969) showed reasonably good agreement between theoretical and experimental results. Monkmeyer et al. (1974) considered the effect of inertia and developed a new mathematical model that described the bottom withdrawal of a viscous, nondiffusive and linearly stratified fluid. The model can be used to predict withdrawal layer thickness and/or discharged water quality in an impoundment.

24. Bohan and Grace (1973) conducted an extensive laboratory investigation to determine the withdrawal zone characteristics in a randomly stratified reservoir. Water was released through a submerged orifice, over a free or submerged weir, and through a combination of these outlets. As a result of their investigations, generalized relationships were obtained for the vertical limits of the withdrawal zone and the vertical velocity distribution. They also proposed a procedure to determine the vertical flow rate distribution which can be applied as a weighting function to evaluate various water-quality parameters in the reservoir release.

25. One of the cases investigated by Bohan and Grace (1973) was withdrawal over a rectangular weir. A sketch of the variables included in this study is shown in Figure C16. The experimental data were plotted in terms of the densimetric Froude number, F, (Figure C17) and the

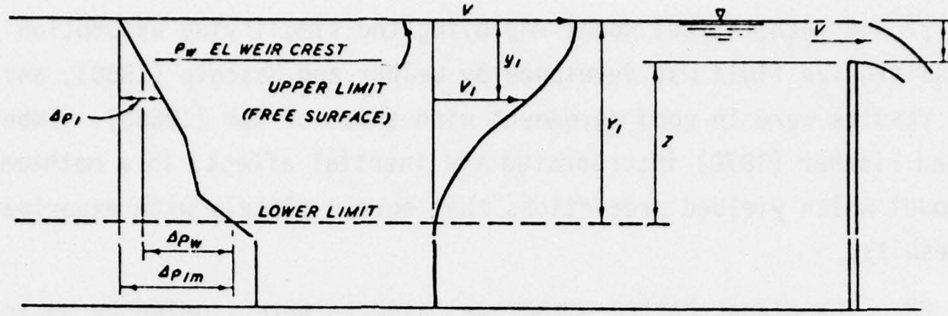


Figure C16. Definition sketch of variables for weir flow

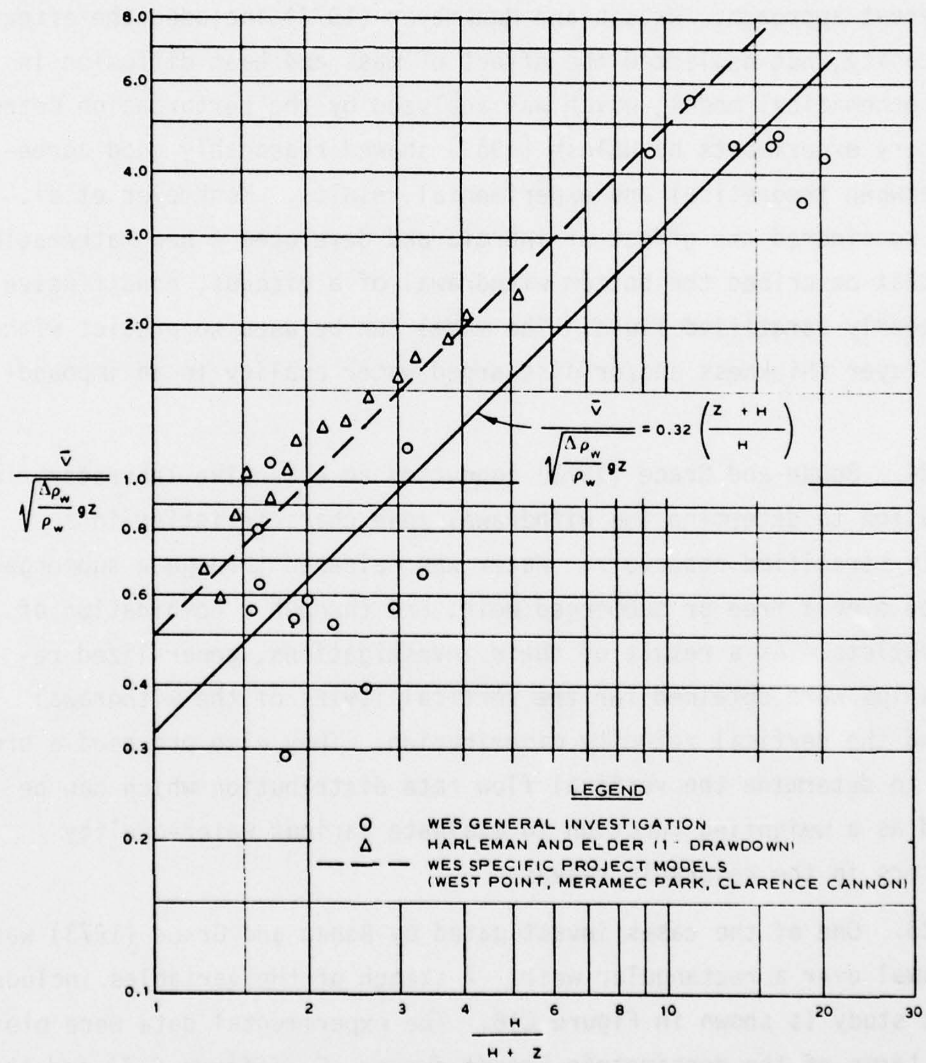


Figure C17. Withdrawal characteristics of weir

ratio of the withdrawal zone thickness, Z, to the head over the weir, H. These data are shown in Figure C17 and can be approximated by a curve with the equation

$$\bar{V} = 0.32 \left(\frac{Z + H}{H} \right) \sqrt{\frac{\Delta\rho_w}{\rho} gZ} \tag{C12}$$

where $\bar{V}$ is the average velocity over the weir (in fps); Z is the vertical distance from the elevation of the weir crest to the lower limit of the zone of withdrawal (in ft); H is the head on the weir for free flow or depth of flow over the weir for submerged flow (in ft); $\Delta\rho_w$ is the density difference of fluid between the elevation of the weir crest and the lower limit of the zone of withdrawal (in g/cc); ρ_w is the density of the fluid at the elevation of the weir crest (in g/cc); and g is the acceleration due to gravity (in ft/sec^2). The data of Harleman and Elder (1965), for which not more than 1 percent of the total flow under a plane skimmer wall was withdrawn from the stratum above the interface of a stratified lake upstream of the wall, are also shown in Figure C17. Recommendations published by Bohan and Grace (1969) were used by Fruh and Mäsch (1972) to predict the water-quality parameters of the releases from Lake Livingston.

26. One of the first extensive theoretical analyses of flow of layered fluids is attributed to Craya (1949), who used conformal mapping to determine the withdrawal characteristics of a two-layered fluid. Later, Wood and Lai (1972) and Lust and Wood (1974) conducted theoretical and experimental studies along the lines followed by Craya.

27. Although the hydraulics of selective withdrawal are not very well understood, the concept has been applied extensively in the United States. A complete register of selective withdrawal works in the U. S. for the pre-1970 period was prepared by the ASCE "Task Committee on Outlet Works, Committee on Hydraulic Structures."

APPENDIX D: ECONOMIC ANALYSIS

Basin Area and Shape

1. The cost for constructing a disposal area depends primarily upon (a) the area enclosed and (b) the shape of the dikes. For a unit area, the costs will be assumed to be proportional to the total length of dikes required, which is equal to the area perimeter, P_a. Consider the model area shown below in Figure D1.

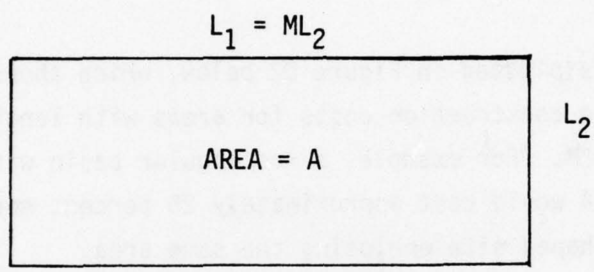


Figure D1. Model containment basin

The length, L_1, can be described as some multiple of the width, ML_2. The area is equal to $L_1 \cdot L_2$ or ML_2^2. For a constant area A, then:

$$L_2 = \sqrt{\frac{A}{M}} \tag{D1}$$

The perimeter, P_a, is given by:

$$P_a = 2ML_2 + 2L_2 = 2L_2\ (M + 1) \tag{D2}$$

By substituting Equation D1 for L_2, we obtain:

$$P_a = 2\sqrt{A}\left(\frac{M + 1}{\sqrt{M}}\right) \tag{D3}$$

The minimum P_a for a constant area, A, is at M = 1, which is a square shape ($L_1 = L_2$). (Circular-shaped areas are not considered to be practical).

2. The relative cost, C_R, for any shape area with respect to a square shape is obtained by substituting M = 1 and taking the ratio $P_a(M)/P_a(1)$ which gives

$$C_R = \frac{M + 1}{2\sqrt{M}} \tag{D4}$$

This function is plotted in Figure D2 below, which shows the relative increase in dike construction costs for areas with length-to-width ratios equal to M. For example, a rectangular basin with a length-to-width ratio of 4 would cost approximately 25 percent more to construct than a square-shaped site enclosing the same area.

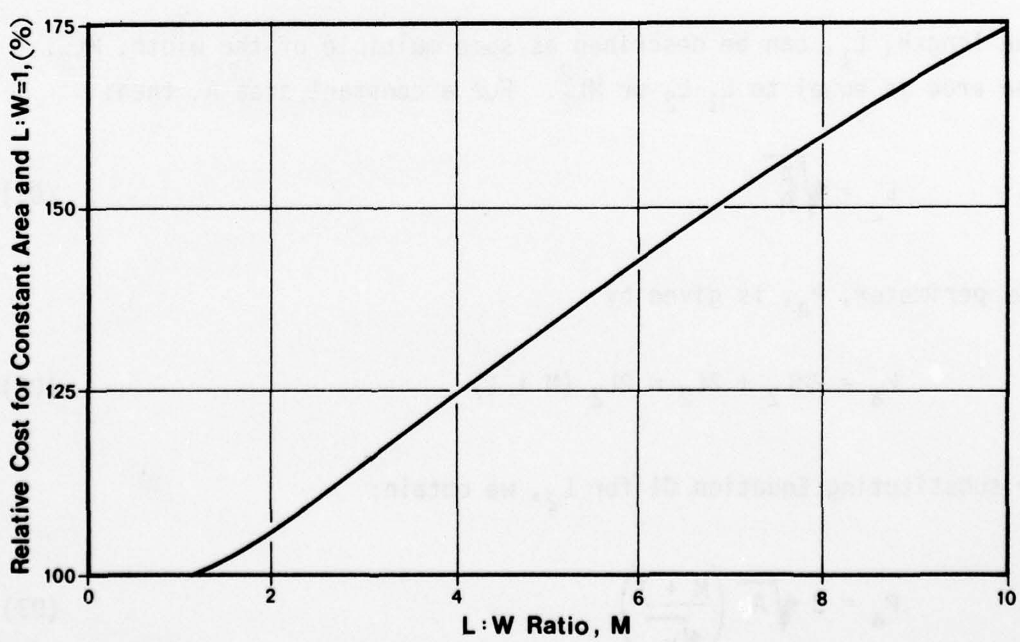
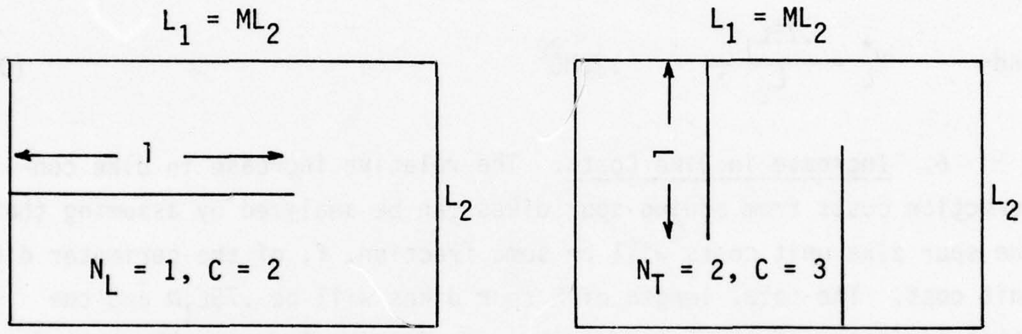


Figure D2. Increase in containment area cost as a function of length-to-width ratio

Internal Configurations

3. The existing length-to-width ratio, M, of a basin may be modified by constructing internal dikes which change the flow patterns and their effective lengths and widths. For example, the model basin of Figure D1 may be modified with one or more spur dikes of number N and length l, to form C = N + 1 compartments as shown in Figure D3.



a. Longitudinal spur dike addition b. Transverse spur dike addition

Figure D3. Model basins modified with spur dikes

It is desirable to describe the modified length-to-width ratio, M*, and the relative increase in dike construction costs and other factors in terms of the existing length-to-width ratio, M, and the number of spur dikes to be added, N. The following analyses illustrate how this can be accomplished for either generalized or specific cases.

Longitudinal Spur Dikes

4. Longitudinal spur dikes are placed parallel to the long dimension of the basin (Figure D3a) and their length, l, will be set equal to $.75L_1$, as explained in Part IV. The effects of adding N spur dikes on the modified length-to-width ratio, M*, and on the relative increase in dike costs are analyzed below, along with the resulting loss in usable area and volume of the basins.

5. Modified Length-to-Width Ratio. The new length-to-width ratio, M*, is equal to L_1^*/L_2^* where L_1^* and L_2^* are the modified flow length and width, respectively. The modified flow length for longitudinal dikes will be equal to $.75L_1$ times the number of flow compartments C; and the modified width will be equal to the original width divided by the number of compartments, C. Therefore:

$$L_1^* = .75L_1C, \qquad L_2^* = L_2/C$$

and

$$M_L^* = \frac{.75L_1C^2}{L_2} = .75MC^2 \tag{D5}$$

6. Increase in Dike Costs. The relative increase in dike construction costs from adding spur dikes can be analyzed by assuming that the spur dike unit costs will be some fraction, f, of the perimeter dike unit cost. The total length of N spur dikes will be $.75L_1N$ and the total perimeter of the outside dikes is given by Equation D2; therefore:

$$\text{Increase in dike costs, percent} = \Delta C_L = \frac{.75L_1N(f)}{2L_2\,(M+1)} \cdot 100$$

$$= \frac{.375MN(f)}{(M+1)} \cdot 100 \tag{D6}$$

7. Loss of Area. The loss of effective area, ΔA_L, caused by adding spur dikes will be based on a spur dike width at the water surface of W_s, and a total basin surface area, A; therefore;

$$\text{Area lost, percent} = \Delta A_L = \frac{.75L_1N(W_s)}{A} \cdot 100 = \frac{.75ML_2N(W_s)}{A} \cdot 100$$

By substituting Equation D1 for L_2, we obtain:

$$\Delta A_L = \frac{.75MN\sqrt{A/M}\,(W_s)}{A} \cdot 100 = .75N(W_s)\sqrt{M/A} \cdot 100 \tag{D7}$$

8. Loss of Volume. The loss of basin volume, ΔV_L, caused by the spur dikes will be based on a unit volume per foot of spur dike, V_s, that is below the water surface; and a total basin volume, V_B. Therefore:

$$\text{Volume lost, percent} = \Delta V_L = \frac{.75L_1N(V_s)}{V_B} \cdot 100 = \frac{.75ML_2N(V_s)}{A \cdot D_p} \cdot 100$$

By substituting Equation D1 for L_2 we obtain:

$$\Delta V_L = \frac{.75MN\sqrt{A/M}}{A \cdot D_p} (V_s) \cdot 100 = .75N \frac{(V_s)}{D_p} \sqrt{M/A} \cdot 100 \qquad \text{(D8)}$$

Transverse Spur Dikes

9. Transverse spur dikes are placed parallel to the short side of the basin (Figure D3b) and their length will be set equal to $.75L_2$. L_1^* becomes $.75L_2C$ and L_2^* becomes ML_2/C. By using procedures analogous to those presented for longitudinal spur dikes, the following equations can be derived:

$$M_T^* = \frac{.75C^2}{M} \qquad \text{(D9)}$$

$$\Delta C_T = \frac{.375N(f)}{(M+1)} \cdot 100 \qquad \text{(D10)}$$

$$\Delta A_T = \frac{.75N}{\sqrt{MA}} (W_s) \cdot 100 \qquad \text{(D11)}$$

$$\Delta V_T = \frac{.75N(V_s)}{\sqrt{MA} \cdot D_p} \cdot 100 \qquad \text{(D12)}$$

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Numerical Example

10. In order to numerically illustrate the analytical concepts previously presented, the following example is given. Consider the case shown in Figure D3a where one longitudinal spur dike is added to a basin of area A. Let the original length-to-width ratio, M, be equal to 2. The modified length-to-width ratio, M*, can be found by Equation D5 for N = 1, C = N + 1 = 2.

$$M^* = .75\ MC^2 = .75(2)(2)^2 = 6$$

11. The remaining analyses depend on the specific characteristics of the dikes. In Figure D4 are shown a representative spur dike and a larger perimeter dike. Both are 7 ft high with a 2-ft freeboard; i. e. the ponding depth, D_p, is 5 ft. The spur dike has a 1:2 slope while the perimeter dike has a 1:3 slope. The total unit volume of the spur dike is 112 ft^3/ft or about half that of the perimeter dike, which is 217 ft^3/ft. The width of the spur dike at the water surface (W_s) is 10 ft and its unit volume (V_s) below water is 100 ft^3/ft.

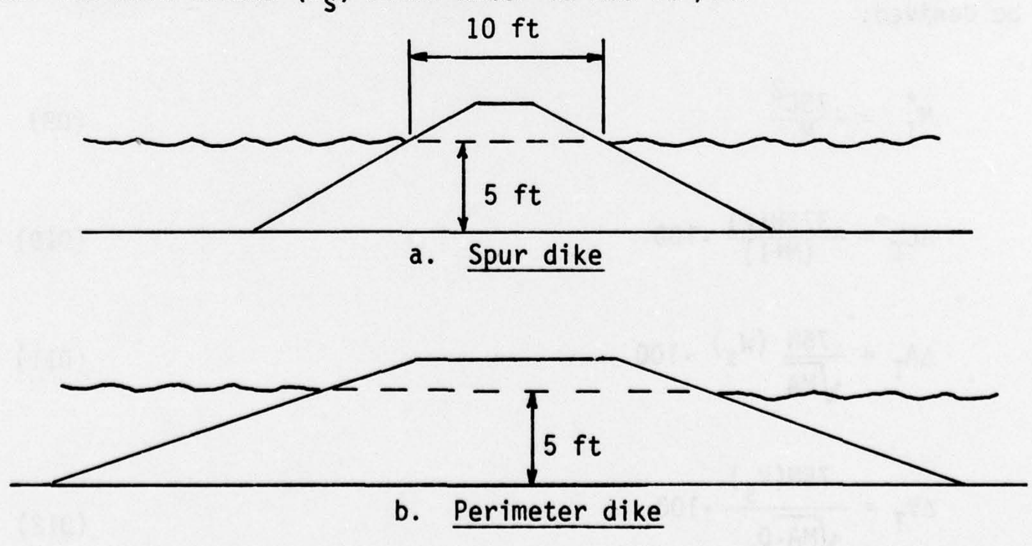

Figure D4. Representative spur dike and perimeter dike

For this example, the unit cost for building the spur dike would be about half that of the main perimeter dike since proportionally less volume of material is required per lineal foot. This does not consider that the construction methods and costs for building the spur dike (such as hydraulically) could be less expensive per cu yd than those for the perimeter dikes. Therefore the coefficient f in Equation D6 would be 0.5 maximum, and Equation D6 would reduce to

$$\Delta C_L = \frac{.375(2)(1)(0.5)}{(3)} \cdot 100 = 12.5$$

resulting in an estimated 12 percent increase in construction costs by adding one longitudinal spur dike.

12. For a ponding depth, D_p, equal to 5 ft, the dike width at the water surface (W_s) is 10 ft, and the unit volume below water (V_s) is 100 ft^3/ft; therefore Equations D7 and D8 reduce respectively to:

$$\Delta A_L = 7.5\ N\ \sqrt{M/A} \cdot 100 = 10.6/\sqrt{A} \cdot 100$$

$$\Delta V_L = 15\ N\ \sqrt{M/A} \cdot 100 = 21.2/\sqrt{A} \cdot 100$$

Note that for the example chosen, the loss in volume is twice the loss in effective surface area. However the relative losses in percent of the available area and volume decrease as the square root of the basin area increases; i. e. these losses become increasingly less significant as the basin becomes larger. For instance, if the basin area is 20 acres (871,200 ft^2), then the area and volume losses are:

$$\Delta A_L = 10.6/\sqrt{871{,}200} = 1.1 \text{ percent}$$

$$\Delta V_L = 21.2/\sqrt{871{,}200} = 2.2 \text{ percent}$$

However, if the basin area is 100 acres (4,356,000 ft^2), the same losses are reduced to 0.5 percent and 1.0 percent, respectively, even though the spur dike is proportionally longer for the larger basin.

13. It can be concluded that the addition of one longitudinal spur dike of length $.75L_1$, and other characteristics as described, when added to a basin with a length-to-width ratio of 2 will (a) increase the effective length-to-width ratio to 6, (b) increase the overall dike costs by 12 percent, (c) cause a maximum loss of surface area of about 1 percent and (d) cause a maximum loss of basin volume of about 2 percent if the basin is 20 acres or larger in size.

14. Equations D5 through D12 have been programmed and solved for different combinations of N number of spur dikes added to basins with various length-to-width ratios, M. The results have been presented as Tables 4 and 5 in Part VI as planning guidelines. The assumptions used for providing representative cases are listed in Part VI but they are identical to those presented in the numerical example given above. If necessary, any significant deviation of a real situation from the assumptions described can be analyzed by disregarding Tables 4 and 5 of Part VI and solving the generalized equations presented in this appendix for the specific case parameters.

APPENDIX E: NOTATION

a	Long side length of rectangular shaft weir
A	Area
A_d	Area of disposal site within dike centerlines
A_e	Area of effective settling within disposal site
A_p	Area of ponded water within disposal site
b	Short side length of rectangular shaft weir
B	Channel width approaching weir
c	Drag coefficient for determining wind stress
c'	Discharge coefficient as a function of weir shape
C	Number of flow compartments created by spur dikes
C_c	Coefficient of weir contraction
C_D	Partially combined discharge coefficient of weir contraction
C_D'	Totally combined overflow coefficient of weir contraction
C_i	Constant transport stream function
C_R	Relative cost of elongated dikes with respect to dikes enclosing square-shaped area of same size
d	Depth of fluid in reservoir
d_p	Pipeline diameter
D_f	Depth of freeboard in diked containment area
D_p	Depth of ponded water in real basin
D_s	Thickness of settled solids layer in disposal area
E	Effectiveness of disposal area in retaining suspended solids
f	Fractional spur dike unit cost relative to perimeter dike unit cost per lineal ft
F	Densimetric Froude number
g	Acceleration of gravity
G	Discharge correction factor for broad-crested weirs
h	Depth of water above weir crest; average depth of water in model basin
H	Head; vertical distance
H_d	Height of containment dikes
H_e	Effective head
H_1	Total head (measured plus effective velocity head)

J_1, J_2	Unspecified functions of y for solution of transport stream function
k	Number of horizontal divisions of basin to partition flow rate
k_B	Effective weir length correction as a function of weir contraction
K	Discharge correction factor for approach flows nonperpendicular to weir
K_o	Discharge correction factor for contracted weirs as a function of weir shape
K_1, K_2	Unspecified functions of x for solution of transport stream function
l	Length of spur dike
L	Length of weir crest
L_e	Effective length of weir crest
L_1, L_2	Lengths of long and short sides of model basin, respectively
L_1^*, L_2^*	Length and width of modified flow path by adding spur dikes, respectively
m	Mesh size in y direction for dispersion program
M	Length-to-width ratio of containment area
M^*	Length-to-width ratio of modified flow path in containment area by adding spur dikes
M_L^*, M_T^*	Length-to-width ratio of modified flow path in containment area by adding longitudinal or transverse spur dikes, respectively
n	Mesh size in x direction for dispersion program
N	Number of spur dikes
N_L, N_T	Number of longitudinal and transverse spur dikes, respectively
p	Pressure
P	Height of weir above settled solids
P_a	Perimeter of containment dikes around disposal area
Q	Volumetric flow rate
Q_i	Influent flow rate
Q_L	Discharge over a labyrinth weir
Q_N	Discharge over a linear weir of same length as labyrinth weir
Q_o	Effluent flow rate
r	Radius of circular shaft weir

R	Width of rectangular sharp-crested weir corresponding to a polygonal weir
RF	Retention factor
S_i	Concentration of suspended solids in influent
S_o	Concentration of suspended solids in effluent
t	Time
T	Thickness of weir plate
T_r	Predicted retention time based on dispersion program
u	Velocity component in x direction
u_o	Surface velocity component in x direction
$\overline{u}$	Average velocity component over a vertical section in x direction
U	Horizontal transport stream function in x direction
U'	Flow velocity far upstream of selective withdrawal sink
v	Velocity component in y direction
v_o	Surface velocity component in y direction
$\overline{v}$	Average velocity component over a vertical section in y direction
V	Horizontal transport stream function in y direction
$\overline{V}$	Average velocity of water flowing over weir
V_a	Approach velocity to weir
V_B	Volume of contained water in basin
V_p	Slurry discharge velocity
V_s	Unit volume per lineal foot of spur dike below water surface
V_w	Wind speed measured 10 meters above water surface
$\lvert V \rvert$	Surface drift velocity
w	Velocity component in z direction
W_s	Width of spur dike at water surface
x	Horizontal distance coordinate of model basin
y	Horizontal distance coordinate of model basin
Y	Number of vertical divisions of water column to partition flow rate
z	Vertical distance coordinate of model basin
Z	Vertical distance from weir crest to the lower limit of the withdrawal zone

α	Angle between inclined weir and vertical
β	Angle between center of flow and weir crest
ΔA_L, ΔA_T	Loss of area caused by longitudinal and transverse spur dikes, respectively
ΔC_L, ΔC_T	Increase in total dike construction costs by adding longitudinal and transverse spur dikes, respectively
ΔV_L, ΔV_T	Loss of volume caused by longitudinal and transverse spur dikes, respectively
$\Delta\rho$	Density difference
$\Delta\rho_w$	Density difference between water at weir crest elevation and bottom of the withdrawal zone
ε	Coefficient of eddy viscosity
η	Change in water surface elevation due to wind
κ	Discharge correction factor for flows nonperpendicular to weir
ρ	Density
ρ_a	Mass density of air
ρ_o	Density of fluid at the reservoir bottom
ρ_w	Density of fluid at the weir crest
τ	Wind stress of water surface
τ_x, τ_y	Wind stress components in x and y directions, respectively
ψ	Transport stream function

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References: p. 95-100.

1. Computer programs. 2. Containment areas. 3. Dredged material disposal. 4. Flow characteristics. 5. Hydraulic efficiency. 6. Mathematical models. 7. Sedimentation. 8. Waste disposal sites. I. United States. Army. Corps of Engineers. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; D-78-12.
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